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JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

THE NEXT JOURNAL AND VOL. 26 INDEX

THIS ISSUE of the JOURNAL concludes Vol. 26 of the *Proceedings*.

The next JOURNAL will be issued in September, beginning *Proceedings*, Vol. 27.

An index of Vol. 26 (JOURNALS 1-8, Nov. 1929 to June 1930) is in preparation. It will be published with a title page and sent to all members and regular JOURNAL subscribers as soon as possible. Do not have your JOURNALS bound at once. Delay the binding for title page and index. Abstracts will be separately indexed.

NEW COMMITTEES AND PROGRAM

EARLY in May the Board of Direction approved a number of new committee projects as recommended by the Advisory Committee.

Announcements of the new tasks will be made in the September *News Letter* when organization of necessary personnel has progressed further.

The Board has also approved the Program Committee's tentative outline for the 27th Annual meeting to be held in Milwaukee, Feb. 24-26, 1931. Work on that outline should make announcements possible in September.

DISCUSSING DISCUSSION

A letter to the Editor sometimes has a valuable thought in it which cannot be published with the name of the writer; this for instance:

"* * * Personally, I get impatient with this situation. Different men write on the same subject without paying any attention to each other or explaining whether they differ at all from what has been done before. This helps neither the writer nor the reader. To the designer it is hopeless. I recognize the personal difficulties involved —that men would sooner appear to present original material than merely to discuss the work of others and also

that they often hesitate to state their points of difference with others—but I think the traditions of A. C. I. are favorable to constructive development of a subject."

How to Run a Convention

Lieut. Comdr. Ben Moreell, (C. E. C.) United States Navy is a comparatively new and very welcome member of the Institute though for quite some time an active student of concrete. He attended the 26th annual convention and was helpful in discussion. The secretary was grateful for certain of his friendly criticisms and in response to specific request he has written recording some of his reactions to the 26th annual program and attendant circumstances. The secretary's letter to which Lieut. Comdr. Moreell refers is of no consequence, except perhaps in its joking suggestion that conventions might be somewhat improved by means of rehearsals. And now that "convention" and "program" are on the carpet, let's hear from other members. —EDITOR

I REALIZE that any undertaking of this kind is beset with difficulties in that, although the participants may enjoy the preparation of papers and discussions more than any other work, their livelihood lies elsewhere. This means that convention work must take a place of secondary importance with the majority of the participants, and it is frequently handled in a very casual and sometimes in a careless manner.

With the hope that some benefit might be derived from the ideas of one who is on the outside looking in, I have set down the following random thoughts.

In my opinion, the most important thing to assure the success of any convention is to provide for the physical comforts of the attendants. By this I mean that complete arrangements should be made in advance to provide a meeting place with the proper conditions of heat, light, fresh air, quiet, and above all, comfortable seats.

I have attended a number of conventions and I have been struck with the uniformity of the degree of discomfort attendant upon sitting in the standard convention chair for one or more hours. I have sometimes suspected that the straight jacket and the standard convention seat were invented by the same man. I am sure that there must be some relationship. When I attend a convention lecture it is because I am genuinely interested in the subject, and still I must confess that no matter how interesting the discourse I usually find myself, after the first half hour, thinking of a well upholstered chair in the lobby.

At New Orleans when we filed into the convention hall for the opening session it was dark, every window was closed, and in spite of the fact that the air outside was cool and balmy, the place was like an oven and the air was thick. The seats were standard.

I cannot conceive of a professional showman handicapping himself by such uncomfortable conditions, nor can I understand why engineers and those associated with engineering activities are averse to the adoption of the practices of the showman.

As you point out in your letter, if you could have a rehearsal of the convention program you could undoubtedly do much better, but this is impracticable. Therefore, the next best thing, as I see it, is to lay out a definite schedule of starting and stopping times for all events on the

program. The convention itself should start on time, whether the majority of the people are there or not. Every event should be scheduled. Each speaker should have his allotted time. The discussion, where practicable, should be prepared in advance by inviting certain individuals, competent to discuss the subject, to prepare their remarks to fit in with the time schedule. Then, at the end, you could have a certain fixed time allotted for impromptu discussion.

If the paper with its attendant discussions does not take up the time allotted to it, the remaining time can be utilized as a recess period, or to pass refreshments, or in some similar manner which will break the monotony of technical discussions. In no case should the paper with its attendant discussions be allowed to run on interminably, as did some of the papers at New Orleans.

To cover the contingency where the writer of a paper does not appear, would it not be practicable to arrange in advance with several of those who can be counted on to attend to have "standby" papers, all prepared and ready to be read in an emergency?

It seems to me that, with the standing that the Institute has attained in the professional world, it should not be difficult to obtain speakers who are willing to abide by rules of this kind which, after all are in the interest of the majority of the attendants, although they may cramp the style of the individual.

With respect to the choice of a convention city, it seems to me that too much weight is often given to the importunities of the local Chamber of Commerce or Kiwanis Club, or what not. After all, the purpose of the convention is to get the members together to exchange ideas, to discuss, and to become better acquainted. If this can be done conveniently and effectively and, at the same time, so as to afford the members an opportunity to enjoy the sights of a new and interesting place, well and good. If not, all other objects should be subordinated to the primary one noted above. If absolutely necessary for the convenience of the greatest number of active Institute members, there should be no objection to having the convention in the same city every year.

Please understand, Mr. Whipple, that the foregoing remarks are not intended to be critical. As I have told you in previous correspondence, my convention experience is very limited. I have personally managed two small conventions, both of them being the annual convention of engineers of the Republic of Haiti. They were run on a rigid time schedule, but then, as you probably know, the country was under martial law and it was easy to exercise military control. Americans are usually less tractable.

B. MOREELL.

The secretary of the Institute feels that while Mr. Moreell's letter has so much in it of value it should be passed on to the membership, it calls nevertheless for some comment.

A sense of physical well-being in the audience adds much to the talent and the wisdom of the speaker. There is no question about that, nor is there any doubt that the "standard convention chair" is an abomination—made more abominable by a bare and noisy floor.

The question is and the Secretary presents it to the membership seriously: Should the first problem be to select

a meeting room that promises fresh air and comfort or should it be to select "a city" where advantage will be taken of the best available meeting accommodations, on the assumption that one factor of interest offsets another of comfort?

The "showman" is to be emulated within the limits set up by the fact that contributors to engineering society programs are not "show people" and that, presumably, members of such a society are more interested in "the meat in the package" than the spangles with which the showman would insist on having it bedecked. It is inconceivable that a technical society would keep a valuable contribution off the program because its presentation wouldn't pass muster before the showman's "director."

Contributors to programs might however with great triumph for themselves tell only so much of their stories as will leave many questions to be asked. The convention speaker who attempts to tell *all*, overwhelms his listeners and leaves his audience no part to play. The convention speaker who leaves an unsurfeited appetite, leaves himself an "encore"—for answering questions. A convention session is usually most successful in which the greatest number are aroused to participation—not numbed into silence. An eager student may be satisfied to *read* every detail (if not he can skip over some paragraphs) but a listener is either helpless or impolite.

The Institute has tried the time schedule and warning signals but chairmen are often too tender hearted to interrupt speakers who are so interested in their experience that they are oblivious of a restless audience.

The nub of the matter seems to be in the fact that some of the most valuable contributors to our technical literature are not the best platform performers and also that technical men generally are rather too cautious in filling all the gaps in their procedure, which takes a good deal of time. Completeness in the written record becomes a burden in the spoken word. An important convention purpose is to stir up reactions out of diverse experience and to put such a polish on the facts as only the rub of discussion can create. It might be set up as a principle of convention-making that no number is worth more time than its discussion will consume. A 30-minute paper that prompts only a 10-minute discussion is not a good convention paper although it may become a classic as published.

It might be possible to get able and goodnatured Institute members to write papers to carry in their pockets to the conventions so they could serve as pinch-hitters if a scheduled performer didn't arrive, but the Secretary is not too hopeful.

There is still much to be said on "How to Run a Convention". Let some others say their say.

A DELEGATE TO JAPAN RETURNS 'ROUND THE WORLD

A LETTER FROM WILLIAM K. HATT*

THE World Engineering Congress at Japan stands out among the three International Congresses that I have attended as of signal technical merit with respect to the scientific and engineering papers presented, and, also, in the extraordinary skill with which our Japanese colleagues conducted a complicated schedule of social functions and inspection trips.

*Head School of Engineering, Purdue University; A. C. I. delegate to World Engineering Congress.

After completing my journey around the world the charming country of Japan is alive in my memory, not only because of the uniform courtesy of its people, and the picturesque attraction of their customs, and the beauty of landscape, but, also, because of the comfort of travel where modern hotels and efficient transportation leave the traveller in a mood to enjoy Japan, and the hospitality of the people. Japan engineers are to be admired for what they have done in their profession, and especially for completing the enormous program of post-earthquake construction.

DURABILITY OF CONCRETE

A LETTER FROM B. MOREELL*

TO ONE who is interested in all types of concrete structures, it is a striking fact that practically all of the tests which have been devised to determine comparative qualities of concretes are based on considerations of relative strengths, and, more striking still, that these relative strengths are determined at comparatively early ages. We appear to say, in effect, that if the concrete will last for 28 days and have adequate strength at that age, it will last forever.

Where we are dealing with concrete structures exposed to severe weathering conditions and to active chemical agents, we know that the strength at early ages is not a true indicator of its ultimate resistance to deterioration. We can go still further and say that the generally accepted standard physical tests and chemical analysis for portland cement are of little or no value in determining the relative durabilities of the resulting concretes under severe exposure conditions. This fact has been conclusively proved by the experiments of the Portland Cement Association and those of the Department of Agriculture.

It appears that the desire to meet the requirements for high strengths at early ages of the largest individual user of portland cement—the road builder—has influenced the cement producer and the technical societies dealing with cement and cement products, to place a preponderant emphasis upon strength requirements. It should be noted, however, that concrete is rapidly becoming an increasingly important factory in other types of structures where exposure conditions are much more severe than in roads. Among these are river and harbor works, irrigation and drainage structures, bridges, dams, etc. Many of these are of a character which attracts public attention and where a failure will exert a very important influence on public opinion in general and on engineering thought. It is conceivable, in such a case, that a failure to resist weathering action will result in prejudice against concrete, and will cause its rejection even for structures where there can be no question of its superior qualities. It is evident that even though structures requiring highly durable concrete now constitute only a small portion of the total, we can ill afford to neglect the effect which the failure of such a structure will have on public opinion. If public confidence in the durability of concrete is once destroyed, the use of that material for large engineering projects will be greatly reduced.

The durability of a concrete structure is dependent upon two factors which are, (a) the procedures involved

*Lieut. Comdr. (C. E. C.) U. S. N., Bureau of Yards and Docks.

in the fabrication of the structure, and (b) the characteristics of the ingredients of the concrete. The first of these involves correct principles of design and proper procedures in proportioning, mixing, conveying, placing, and curing. Much progress has been made along these lines, and thanks to the excellent work of the Portland Cement Association and other investigators, there is good reason to believe that better procedures will continue to be developed from day to day.

With respect to the choice of proper ingredients, the situation is not so satisfactory. There are a great many unanswered questions, which can be summarized by asking, "what variations can be made in the characteristics of the water, fine aggregate, coarse aggregate, and cement, without adversely affecting the durability of the concrete and its ability to meet the exposure conditions for which it is designed?" The difficulty is that heretofore investigators have based their conclusions almost entirely on the results of strength tests. For example, the most complete investigation of the use of impure waters in concrete is that of Prof. Duff A. Abrams published in the Proceedings of the American Concrete Institute, 1924. These are very valuable data, but the conclusions as to the suitability of impure waters are based entirely on compression tests of plain concrete cylinders.

We should know more about the effect of variations in the ingredients on the ultimate resistance to corrosion of the concrete. Should impure water be used for concrete which will be exposed to sea water? Can it be safely used for reinforced concrete, and under what conditions? Is coral sand suitable for a concrete exposed to alkali action? Can crusher screenings be used for such a concrete and, if so, should the fine dust be removed, and to what extent? Would a mixture of crusher screenings and coral sands be better than either used alone? Can we devise some more definite indication of durability than the sodium sulphate soundness test for coarse aggregate? Are surface treatments necessary or desirable to increase durability? When are protective veneers—brick, granite, creosoted plank, etc.—justified? Are admixtures of any utility? Does hydrated lime have a definite deleterious effect on concrete exposed to sea water?

These are a few of the many questions which present themselves at this time. I have my own opinions on each but I must confess that they are based on considerations of the most general nature and, in most instances, I have no definite experimental data which I can use as a foundation for my judgment.

In the foregoing I have not mentioned the cement. This is a subject on which there has recently been much discussion. In our eagerness for standardization, there has been a very strong aversion to the use of more than one standard portland cement. Perhaps there are economic reasons for this which are apparent only to the cement producer. As an engineer, I must say that I cannot see the soundness of this position.

Where would the steel industry be today in competition with other building materials if they had held to one standard of quality? We must recognize that the conditions of service which will be met by cement used in

the construction of a large irrigation canal in alkali soil are entirely different from those met in the ordinary concrete frame building. It is conceivable that the properties which the road builder is looking for in cement are not the same as those which I want in a cement to be used for a large dry dock which will be exposed to water heavily contaminated with sewage and to large extremes of temperature. Why then should we attempt to stretch our specification to a point where it will meet the requirements of these widely divergent uses? If we do this it is inevitable that we will fail to meet the requirements of any one user to the best advantage. Would it not be better if we had a specification for a cement to be used for sea water, alkali earth, and similar exposure conditions, one to be used for highway work, and another, let us say, for ordinary building construction? Certainly, greater care in the selection of raw materials and in the manufacturing processes should be exercised in making a cement which will be used in a waterfront structure exposed to severe conditions than if the cement is to be used in a concrete building.

It is a fact that at the present time many cements which meet all of the A. S. T. M. specification requirements differ widely in their qualities of durability.

This question of durability is the most important which now confronts concrete investigators. There are few instances on record where the failure of a concrete structure can be directly attributed to a deficiency in the strength of the cement. Notable failures have resulted either from faulty design or from improper construction procedures, or more frequently, from the lack of durability in the concrete.

You are no doubt familiar with many reports on this subject, among which are those of the National Research Council, the San Francisco Bay Marine Piling Committee, and the Institute of Civil Engineers. These have furnished valuable information, but little accurate experimental data. In my own experience, I have seen many waterfront structures in advanced stages of deterioration, concrete sewers which have disintegrated badly, drainage works which have failed, etc. A massive structure recently completed, which was built under the most careful supervision and in accordance with the best practice in construction procedures, has, within a period of 18 months, shown indications of deterioration of the concrete. A monumental building with which the utmost care was taken shows large areas of efflorescence on one of the concrete walls.

What is the answer? We must give more attention to this matter of durability. We need more research. But the specific object of this research should be *durability*, not strength. We must develop sound construction procedures and tests for materials so that the practicing engineer can design his structure for a definite degree of durability just as he now designs it for a definite strength. When this is accomplished it will no longer be necessary to resort to the use of expedients (admixtures, surface treatments, special cements, veneers, etc.) of doubtful value and possible harmful effect to obtain a concrete which will successfully withstand the conditions for which it is designed.

IN CONSTANT REPAIR

(Conversation with Dr. Sam'l Johnson)
Dr. Johnson: A man, Sir, should keep his Friendships in a state of Constant Repair.
The Author: And no less, Sir, his Information.

6—NEW RECIPES

BY ARTHUR R. LORD

PORTLAND CEMENT, having attained in vigorous health, to the hundredth year of its use, may properly regard the numerous special cements, now competing with it for our favor, as upstarts. But it should remember that an adventurous cook eagerly embraces new recipes and frequently tries them out, not on the dog, but on her most important subjects. Is it possible that our rapid conversion to the many "high-early-strength" cements may be a similar gesture of adventure, undertaken with no adequate understanding of its possible effect on our future happiness?

I approach this subject with not a little diffidence. First on any list of special cements, longest in use, best known, comes Atlas Lumnite, pet of our retiring president, Uncle Edward D. Boyer. Who would carelessly expose himself to the shafts of this unmatched champion among the wits and story tellers of our Board of Direction? And when we come to the second on such a list, we find closely associated our new president, Professor Duff A. Abrams! But I have less fear of Duff, having known him before his rise to fame as an ardent "dry." Truly the way of an enquirer in the Institute is not always plain, but having set foot on the path we cannot well turn back.

Lumnite cement is characterized in my mind by three attributes—a tremendous chemical activity leading to strength at the end of hours instead of our usual days, a corresponding generation of heat and thirst for water at early periods, and a muddy brown color which varies over a wide range. I read the voluminous Lumnite advertising literature with some chagrin. It appears so simple to use this material with complete satisfaction—and yet I have had difficulties. I have been called in to help others out of trouble also. It *appears* to remove the need of any protection in cold weather—even in very cold periods—but it has frozen solid for me at 25°F.! Men as dumb as I get into trouble by following too lightly the care-free program implied (possibly not advocated) in these advertisements. It would seem that precautions quite unnecessary for sheltered foundation concrete may be essential for the same concrete exposed high in air to a brisk northwest wind. I would like to know somewhat definitely under what circumstances this valuable material does require protection and just when heat and enclosure may be dispensed with. When may we save and when may we not? Is Lumnite cement more injured by excess mixing water than ordinary portland cement? What is the water-cement-strength-time relation for this cement with different aggregates? How much water is required to cure it properly, to secure maximum strength and watertightness? Probably the ponding methods that I have sometimes used are unnecessary, but where shall I draw the line? If I seem to be over-critical in my suggestions, it is because I believe that a material as long on the market as this has been, should have given users a better basis for its

intelligent application than I, at least, have received. This I say in spite of the fact that we have more technical information on Lumnite than on any other special cement. I have used it with success, but my use has been far more cautious and controlled than the advertising matter would appear to require.

A most excellent case can be pleaded for Incor cement, without tests and without experience. If one burning and one grinding produce excellent portland cement, surely a second burning and a second grinding should produce a very superior product. If ordinary fineness of grinding is essential, is not the extreme fineness of Incor a most valuable asset? It would seem so, and yet an adequate display of confirming tests would be helpful. We know that its early strength considerably exceeds that of standard portland cement, but we could make use of a a modified Abrams water-cement-strength formula for this material for our concrete design. What is the effect of temperature during and after setting on this material? What is the strength-age relation under various curing conditions? I presume that my friend would readily give me any of this data which is available but my point is that such information is essential to the intelligent use of a cement and it should be given to the Institute in a full disclosure. The questionnaire for admixture salesmen, contained in the first article in this series, may be applied as well to special cements, and I invite Lumnite, Incor, Quickard, Prestolith Velo and Super Cement to schedule their data in such a manner as this questionnaire requires. In this way they will convince themselves and their users as to what may be expected of each cement.

With Prestolith Velo and Quickard I have had no use experience. I have received their pamphlets—talked with their salesmen. Both claim 24-hr. strength sufficient for immediate service. Both illustrate examples of their successful use, although they are more recent products than Lumnite and Incor. Super Cement I have used with satisfaction. But on none of these have I anything approaching an adequate supply of information for their intelligent application. What do they claim (and prove) on the score of permanence, volume changes, and the numerous other matters covered by the questionnaire?

We have still a fourth approach to high-early-strength concrete, involving the use of rich mixes and low water-cement ratios, in which Universal Portland Cement Co.* has been the aggressive leader. This application of ordinary portland cement involves some new questions that go beyond our available data. What will be the shrinkage of such concrete? How will such concrete stand up if not afforded proper curing conditions? Time will tell, of course, but some of us would like to use these newer methods and materials with that assurance which only adequate data, in hand in advance, makes possible. I invite Universal Portland to join the ranks of questionnaire answer filers to the end that our information may be kept in a state of Constant Repair.

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*Now as the Universal Atlas Cement Co. this company has assumed responsibility for Lumnite as well.

TRANSVERSE TESTS

FROM A LETTER BY C. M. DUFF*

CONSIDERABLE stress is being laid on the transverse test of concrete beams for determining the modulus of rupture or maximum fiber stress at breaking, and using these results as a criterion as to when a pavement can safely be opened to traffic. I would like to ask whether any of the committees have made a study of the varying results that may be obtained from the same concrete by changing the span length and testing under various types of loading. I have found in testing 6 x 6 in. beams with spans of 28, 18, 14, and 10 ins. where the load has been applied either at the middle or at third points, that as the span is shortened a variation of about 50 per cent is obtained in computing the fiber stress. Evidently there is a minimum span length that may be used that will give a correct value for the Modulus of Rupture. This minimum length span is no doubt a function of the depth of the beam tested. I expect to obtain more information on transverse testing of concrete this summer in connection with a paving project adjoining Lincoln.

Following is a meager summary of the tests made on beams submitted from the field last year. The concrete used on projects 240A, 234A, 215B, and 180 was composed of a single aggregate known as Platte river sand gravel having a uniform grading from 0 to ¾ in. and a fineness modulus of about 4.0. The quantity of cement used was 8 bags per cu. yd. in place. On project 17 coarse and fine aggregates were used. The coarse aggregate was a comparatively soft limestone crushed to a maximum size of 1½ in. The fine aggregate was Platte river sand grading from 0 to ¼ in. The quantity of cement used was 7 bags per cu. yd. in place. On project 49 E&H the aggregate was Platte river sand gravel but only 7 bags of cement per cu. yd. were used. However, this project was put in as a vibrolithic pavement.

All tests were made on a Riehle Universal testing machine and the beam supports were placed on a 2x8 plank having a curved top so as to be self-adjusting for any warp that might be in the beams. The beams were all 6x6x30 in. and the broken pieces resulting from transverse tests were tested in compression after being capped with a neat mortar made by mixing Lumnite cement and plaster of paris in proportions of about half and half, which has the property of obtaining a high early strength in from 4 to 6 hours. The results of these compression tests are also shown on the enclosed summary and may be of some interest. The results of transverse tests for center loading and on 18 in. span are also shown for a beam cast at the same time and from the same concrete.

The results of the laboratory tests made during the winter did not turn out as well as expected, but certainly showed the need of further investigation and study on transverse beam testing. These laboratory tests were undertaken with the idea of finding at what lengths the modulus of rupture would remain constant for any given concrete of the same known mix and curing, and also to obtain some information as to the effect of cold and slow curing on the results of transverse and compressive strength of concrete. Two sets of beams were made using Platte river sand gravel aggregate and 8 bags

*Professor of Applied Mechanics, University of Nebraska.

of cement per cu. yd. Duplicate beams were made in each set and after 24 hrs. half the beams were placed outside for curing. The first set consisted of two beams 6x6x60 in. and two beams 6x6x30 in. The second set consisted of two beams 6x6x72 in. and two beams 6x6x30 in. Concrete cylinders 6x12 in. were also made for compression tests from both batches. Each set was made from only one batch of concrete which was progressively filled into all forms so that there should be no difference in any of the specimens made.

The beams in the first set were broken at the end of seven days and those in the second set were broken at the end of 28 days. Part of the results are given on the enclosed summary of laboratory transverse tests. All tests summarized were made by loading at the third points although some tests were made for center and cantilever loading.

The results of the 7-day tests made on the first set of beams cured in the laboratory appear to indicate a decrease in the modulus of rupture as the span length was shortened, which is just opposite to the results previously obtained on field specimens. The results of the 28-day tests made on the second set of beams, however, show the same characteristics as the previous tests made on field specimens. The tests on the second set of beams are rather incomplete due to the fact that the forms were not stiff enough to warrant moving at the end of 24-hrs. and the long outside beam was cracked in two places while being placed outside for curing. The beam cured inside was also broken while being placed in the testing machine so the longest span length used was 30 ins.

At present I have no explanation to offer for the modulus of rupture in the first set of beams tested at 7 days giving results just opposite to those obtained on the second set of beams tested at 28 days, and on the beams made in the field.

I have read with interest the paper "Flexure and Tension Tests of Plain Concrete" by Gonnerman and Shuman found in the 1928 Report of the Director of Research of the Portland Cement Association. Paragraph (4) of the "Conclusions From Series 209" given on page 182 of the report referred to above most nearly covers the tests we have been making. These conclusions show the flexural strength obtained from cantilever loading and center loading to increase materially as the length of span is decreased, but that practically no effect on the modulus of rupture was found for $\frac{1}{3}$ point loading. The summary of these values was found in Table 16 on page 200 of the report, and also shown graphically in Fig. 30 on page 176.

I am still of the opinion that compression tests give a much more accurate measure of the quality of concrete than do the transverse tests and I question the advisability of selecting a value for the modulus of rupture which may be considered safe strength for opening pavements to traffic. This is especially true where the type of testing machine, method of loading, size of beam and span length have not been standardized.

SUMMARY OF TRANSVERSE TESTS MADE ON BEAMS CAST IN LABORATORY

First Set of Beams

Dates made	January 2, 1930
Dates tested	January 9 & 30, 1930
Method of loading	⅓ Point
Nominal depth of beam	6 inches
Nominal breadth of beam	6 inches
Aggregate used	Platte river sand gravel
Mix used	8 bags of cement per cu. yd. of concrete

TEST RESULTS

Modulus of Rupture, lbs./sq. in.

Span Length, Inches	Cured in Laboratory		Cured Outside	
	7 day	28 day	7 day	28 day
56	309		189	
36	292		155	
18	269	237	184	220
9	231	234	133	253

Second Set of Beams

Date made.....January 3, 1930
Date tested.....January 31, 1930
Method of loading.....⅓ Point
Nominal depth of beam 6 inches
Nominal breadth of beam 6 inches
Aggregate used.....Platte river sand gravel
Mix used.....8 bags of cement per cu. yd. of concrete

TEST RESULTS

Modulus of Rupture, lbs./sq. in.

Span Length, Inches	Cured in Laboratory	Cured Outside
	28 Day Test	28 Day Test
30	275	
24	303	232—234
15	309	
12	421	
9		298

SUMMARY OF TRANSVERSE TESTS MADE ON BEAMS CAST IN FIELD DURING CONSTRUCTION

(8 Bags of cement per cu. yd. except as noted)

1/3 Point Loading

Proj. No.	Date Sampled 1929	Age When Tested Days	Modulus of Rupture, lb./sq. in. 28 in.	Span Lgt. 114 in.	10 in.	Compressive Strength of 6x6x10 in. (approx.) Prisms, lbs./sq. in.
17	7/19	7	480	554	645	2888,2434,2790,2896
17	7/20	4	341	552	587	2458,2268,2424,2124
17	8/19	7	461	646	783	2848,2403,2610,2573
17	8/20	14	373	494	573	3387,2969,2935,3463
17	8/21	28	488	592 (18")	692 (9")	4138,3638, (too short)
240A	9/30	10	391	558	650	3199,3306,3658,.....
234A	8/27	28	473	721	846	4001,3772,3829,3143
215B	8/22	28	534	809	835	3030,2810,3003,3389
180	7/19	12	607	865	1012	4498,3468,3956,3685
49E&H*	8/20	28	528	692	685	4139,5283,4411,5010
			Center Loading			
180	7/25	28	479	825	957	3783,4004,4247,4669

*7 Bags of Cement per cu. yd.

SUMMARY OF TRANSVERSE TESTS MADE ON BEAMS CAST IN FIELD DURING CONSTRUCTION

(8 Bags of Cement per cu. yd. except as noted)

Center Loading

Proj. No.	Date Sampled 1929	Modulus of Rupture, lbs./sq.in. Span Length, 18 inches Age When Tested 4 da.	3 da.	7 da.	10 da.	28 da.
17	7/19	675			671	
17	7/20	545		692		
17	8/19	552			651	
17	8/20	508			555	
17	8/21	562			631	
240A	9/30				446	
234A	8/27	585			910	
215B	8/22		505			610
180	7/19				609	
180	7/25				630*	839
49E&H†	8/20	579			766	

*16 day test.
†7 bags of cement per cu. yd.
These results may be compared with the results obtained with 1/3 point loading for beams made from same concrete.

NEW MEMBERS

APPLICANTS for membership in April have been approved by the Board of Direction as follows:

American Builders Supply Co., 405 Garden St., Louisville, Ky. (Jas. F. McCracken)

Baily, R. W., 1528 Walnut St., Philadelphia, Pa.

Canadian National Railways, Montreal, Que., Can. (W. A. Duff)

Converse & Co., Inc., J. B., Mobile, Ala.

Gunn, Charles M., Gunn, Carle & Co. Ltd., 444 Market St., San Francisco, Calif.

Hoskins, C. H., Southern Portland Cement, Ltd., Kembla Bldg., Margaret St., Sydney, N. S. W., Australia.

Kaiser Paving Co., 1522 Latham Sq. Bldg., Oakland, Calif. (T. M. Price)

Kirst, Leonard R., c/o N. O. Public Service Inc., 317 Baronne St., New Orleans, La.

Los Angeles Harbor Department, Berth 161, Wilmington, Calif. (G. F. Nicholson, Harbor Engr.)

McBurney, J. W., 414 Taylor St., Chevy Chase, Md.

McMillan Contracting Co., Thomas, 26 Court St., Brooklyn, N. Y. (Thomas R. McMillan, Pres.)

Made-Rite Products Co., 5223 McKissock Ave., St. Louis, Mo. (Robert Fullerton, V. P.)

Miller, Vincent, c/o Chirigui Land Co., Puerto Armuelles, Rep. of Panama, C. A.

Morgan, J. Grant, 403 Inter-Island Bldg., Honolulu, Hawaii.

Pasadena Water Dept., Room 319 City Hall, Pasadena, Calif. (S. B. Morris, Ch. Engr.)

Roche, Edward C., 39 Chickatawbut St., Dorchester, Mass.

Rylander, Paul N., Macdonald Spencer Eng. Co., 420 Lexington Ave., New York, N. Y.

Williams, Chas. P., 4259 Witherby St. San Diego, Calif.

The next issue of the JOURNAL will be for Sept. 1930 — beginning Vol. 27 of *Proceedings*. Index and Title pages of Volume 26 are in preparation.

STATEMENT OF THE OWNERSHIP, MANAGEMENT, CIRCULATION, ETC. REQUIRED BY THE ACT OF CONGRESS OF AUGUST 24, 1912

Of Journal of the American Concrete Institute published monthly except July and August at Detroit, Mich., for April 1930.

State of Michigan } ss.
County of Wayne }

Before me, a notary public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the Journal of the American Concrete Institute and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily paper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912 embodied in section 411, Postal Laws and Regulations, printed on the reverse of this form to wit:

1. That the names and addresses of the publisher, editor, managing editor, and business managers are: Publisher, American Concrete Institute, 2970 W. Grand Blvd., Detroit, Mich.; Editor, Harvey Whipple, 2970 W. Grand Blvd., Detroit, Mich.; Managing Editor, none; Business Manager, none.

2. That the owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding one per cent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a firm, company, or other unincorporated concern, its name and address, must be given.) American Concrete Institute, 2970 W. Grand Blvd., Detroit, Mich.

Duff A. Abrams, President, 342 Madison Ave., New York, N. Y.

S. C. Hollister, Vice-President, 311 Elm Ave., Swarthmore, Pa.

Arthur R. Lord, Vice-President, 140 So. Dearborn St., Chicago, Ill.

Harvey Whipple, Secretary-Treasurer, 2970 West Grand Blvd., Detroit, Mich.

3. That the known bondholders, mortgagees, and other security holders owning or holding 1 per cent or more of total amount of bonds, mortgages, or other securities are: (If there are none, so state.) None.

4. That the two paragraphs next above, giving the names of the owners, stockholders, and security holders, if any, contain not only the list of stockholders and security holders as they appear upon the books of the company but also, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduciary relation, the name of the person or corporation for whom such trustee is acting, is given; also that the said two paragraphs contain statements embracing affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner; and this affiant has no reason to believe that any other person, association, or corporation has any interest direct or indirect in the said stock, bonds, or other securities than as so stated by him.

5. That the average number of copies of each issue of this publication sold or distributed, through the mails or otherwise, to paid subscribers during the six months preceding the date shown above is ——. (This information is required from daily publications only.)

HARVEY WHIPPLE

Sworn to and subscribed before me this 24th day of April 1930.

[Seal] ETHEL E. BLAINE

(My commission expires July 12, 1930).

Papers and Reports—In this Issue

Proceedings American Concrete Institute

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Publication of discussion of the contributions in the following pages is scheduled. Please see inside cover pages.

Making and Placing Concrete Revetment Mat, Vicksburg Engineer District*

BY MORRIS W. GILLAND†

Stabilization of caving banks has long been recognized as one of the outstanding problems in man's efforts to curb the Mississippi river. It is not the purpose of this paper to discuss the causes of caving banks, nor their effect upon navigation and flood control, but to describe revetment construction methods now employed in the Vicksburg Engineer District.

Until recent years, mattresses made from willows have been standard for subaqueous revetment. These willows grow on sand bar formations between the levee lines along the river. This supply of willows becomes inaccessible during river stages from somewhat below bankfull to flood. Moreover, as revetment construction programs were enlarged, it was seen that the available supply of willows could not keep pace with increasing needs; a suitable substitute was accordingly sought.

In 1914, funds were allotted and experiments initiated in the several Mississippi river commission districts for independent development of suitable concrete mats. The earliest attempts strove to augment rather than to supplant the willow mat; present results indicate that in all save special situations the concrete mat is equal to and cheaper than the willow and can be made and placed more rapidly. During the construction season of 1929 just completed in the Vicksburg Engineer District, all revetment construction was, for the first time, entirely of concrete.

TYPE OF MAT

The articulated type of reinforced concrete mat is used in the Vicksburg Engineer District. The unit of this type is the mat

*Presented at the 26th Annual A. C. I. Convention February 1930

†1st Lieut., Corps of Engrs., U. S. Army.

section, consisting of 25 slabs cast on a single integral piece of wire reinforcement. Each slab is 3 in. thick, 11 in. wide, and 3 ft. 10 in. long. The reinforcement is a 12 x 12 in. wire mesh. Two No. 12 gage wires are placed in the ends of each mat section to provide a twisted union with next succeeding section as later described in mat-sinking operations. Because of the general overall dimensions of one of these mat sections (4 ft. x 25 ft. or 100 sq. ft.), this unit is commonly called a "square," with a volume of 25 cu. ft.

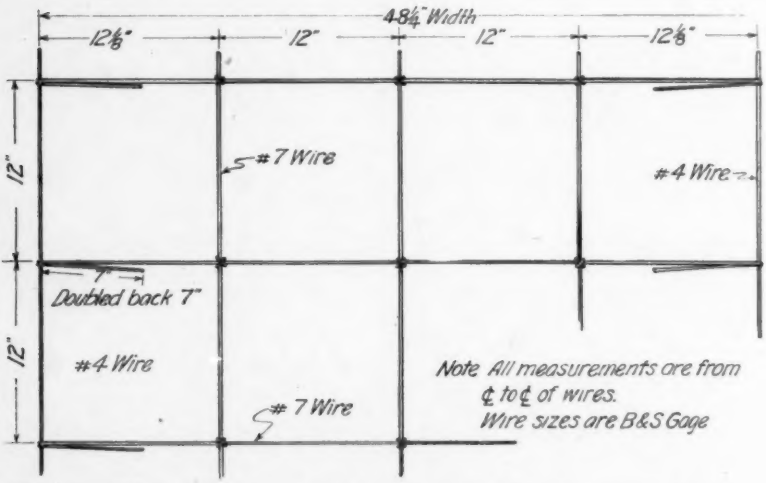


FIG. 1—REINFORCING FABRIC FOR ARTICULATED CONCRETE MATTRESS—FENCE TYPE

LOCATION OF MANUFACTURING UNIT

The unit or field party which manufactures the mat sections is known as the concrete casting plant. It is located on the riverside of the controlling levee, on the southern edge of Greenville, Miss. A spur of the C. & G. R. R. lies along the river bank at the site. Rail operations can be carried on throughout river stages up to 6 ft. over bankfull at Greenville; in other words, several feet above stages that will permit revetment construction.

A steam locomotive crane serves as yard engine and unloads bulky shipments such as wire reinforcement for the casting plant and materials for revetment field construction parties.

MATERIALS

Aggregates: Sand and gravel are from a Mississippi river bar 5 miles north of casting plant site and are washed and separated on the gravel digger at the bar. The gravel has a maximum size of 2 inches. The sand is rather fine; the fines predominate and are uniformly graded, producing a tendency to exude free water on the surface of the concrete shortly after finishing, even though the driest mix consistent with workability is employed.

Cement—The cement used is a U. S. portland cement, standard except that 7-day strength must be attained in 4 days. Shipments are received by rail on the concrete casting plant spur. The cement is handled in cotton sacks from the box cars to covered cement storage barge on a roller conveyor, three sacks at a time on wood pallets, and stacked on the barge by hand.

Reinforcement is received on the casting plant spur track in gondola cars, in rolls 300 ft. long. These are unloaded upon the bank by the locomotive crane and rolled by hand to the waters-edge. They are then unrolled, flattened by passing through a set of three rollers, and cut into 25-ft. lengths with bolt cutters. The wires of the reinforcement (Fig. 1) are drawn from mild copper bearing steel and are heavily galvanized to prevent corrosion. All cross wires and the interior longitudinal wires are No. 7 gage, while the exterior longitudinal wire is a No. 4 gage. All joints are bonded by a mechanical non-slipping wrap. Welding of joints has been found to destroy the galvanized coating of the wires, with rapid corrosion at external joints. Modification of specifications has been considered whereby interior joints may be welded and exterior joints made by mechanical wrap. Ultimate tensile strength of wires is from 65,000 to 80,000 lbs. per square inch. Necessity for fitting forms permits very small tolerance of spacing of wires in reinforcing mesh.

Craft Paper is used to separate successive superimposed layers of mat sections. It is a 50 lb. basis paper, known commercially as "Craft." Oiled, or heavier paper for cold and wet weather has been dropped from consideration, since the doubling of the present paper produces the desired results, i. e., prevention of sticking, with less cost. The paper is shipped in by rail in 200 lb. rolls. These are cut into 25-ft. lengths and rolled up into

small, easily handled tubes in the paper storage barge. This is at present a hand process; investigation is being made into the design of a machine for this operation.

Floating Plant and Equipment—The major items of floating plant and equipment required for a single 10-hour shift with average output of 800 squares per day is as follows:

1 mixer barge
2 cement barges
16 to 24 casting barges (in line)
40 casting barges (in curing storage)
2 shop barges
8 spar and header barges
1 quarterboat
2 small towboats
1 gravel digger
6 to 8 gravel and sand barges
2 gasboat tenders
2000 sections of mat forms
roller conveyors, sack shaker, gasoline barge pumps, etc.

The number of casting barges given above is based on 14 day cure in good summer and fall weather and continuous operation of revetment field construction parties close at hand for steady return of empty barges. At least 20 additional casting barges are desirable to avoid delays when these conditions do not obtain. A brief description of each of the above items of plant follows.

Mixer Barge—The general layout of the mixer barge is shown in Fig. 3. It is of steel, 36 ft. x 120 ft. A covered cement storage barge of standard size houses the cement supply and is lashed to the downstream end of the mixer barge. The cement barge is equipped with two belt conveyors; cement is dumped by hand from sacks upon one horizontal conveyor which lies entirely within the cement house. This conveyor carries the cement to a second conveyor which raises it on an incline to the 11-ton cement hopper on the lower end of the mixer barge. The hopper is equipped with a weighing batcher.

Aggregates are in a sand and gravel barge, tied on the outside edge of the mixer barge. A partition separates the sand from the gravel. Aggregates are transferred from barge to bin by two electric locomotive cranes each with 45 ft. boom and ¾-yd. bucket. The 150-ton sand and gravel hopper is equipped with

weighing batchers, and with steam coils for heating aggregates during freezing weather. A 50 h. p. vertical boiler furnishes steam for the coils, with bunker oil as fuel.

To reduce list or sideroll of mixer barge, metacentric height was kept low, but necessitated a double lift of materials. Mixers are elevated, and aggregates transferred from bin to mixer by two tram dump cars (one for cement and one for sand and gravel) and individual elevating power loaders.

Two 1-yd. concrete mixers operate on staggered cycles and discharge into individual reservoir hoppers. These hoppers discharge into a 1-yd. bottom dump bucket which distributes the concrete to the forms by moving out on a trolley along an I-beam boom. The clearance of the bucket over the forms is regulated by an electric power hoist.

Electric water pumps under the gravel hopper supply water to reservoir and measuring tanks above each mixer. Mixing water is heated by live steam during freezing weather.

An electrically driven air compressor in the engine room operates two sprays for oiling the forms. The sprays, near the boiler, consist of two counter balanced pipes with rubber hose connecting to spray heads in the manner common to many types of automobile filling station air units.

The mixer barge, with its attached cement and gravel barges, is moved along the line of casting barges by up- and downstream wire rope cables which lead out from two electric towing winches.

All units on the mixer and cement barges are electrically operated. Power is furnished by a 225 h. p., 4-cycle, 6-cylinder Diesel engine direct-connected to a 440 volt A. C. generator.

Electric power with its attendant advantages of remote control has been utilized wherever possible. Every machine operator has been placed where he can best view his job. The operator who spots the mixer barge opposite forms on casting barge is placed with his control boxes under the mixers and in prolongation with the boom. The bucket operators ride on a seat on the end of the bucket; one operator regulates rate of travel along the boom and the opening of the bucket dump, the other adjusts the clearance of the bucket over the forms. The electric cranes move

on a slightly elevated track so that the operator may command full view of the gravel barge while rear of crane will clear all persons or machines working on deck of mixer barge.

All electric equipment operates with 3 wire system. Nothing is grounded to the barge; the cranes and tramcars operate from carefully insulated three-rail trolleys.

A 10-K. W. 110 volt, electric generator, driven by pulley and belt from the main generator shaft, provides current for flood lighting the mixer barge and pouring and finishing for night operations. Current for other night operations on the bank and along the line of spar barges (cleaning and placing forms) as well as for the sack shaker, is furnished from Greenville. Five 1000 watt flood lamps and numerous smaller lights at cranes, mixers, batchers, etc., are on the mixer barge. The bank and spar barge lines are 500 watt lamps at an average spacing of 250 feet.

Casting Barges are standard decked scows, 30 ft. x 120 ft. x 7 ft. They may be of all steel, all wood, or composite (steel framed, wood covered) construction. The type of barge construction governs the capacity in squares of mat which may be loaded upon the barge, varying from 250 for all-wood to 300 for all-steel.

Mat sections are cast in tiers with aisles between; the number of tiers per barge and number of mat sections per tier vary with type of barge and location of hatches. The normal loading of a standard steel barge consists of 20 tiers of 15 sections each.

The type of mat, with interior crossmembers on forms, requires complete removal of forms before next mat section can be superimposed; hence the line of casting barges must be sufficiently long to permit final set of concrete, removal, cleaning and resetting of forms prior to next run of concrete. Under average conditions, preparation of forms may be undertaken 4 hours after previous pouring; allowing 1 hour for preparation, each barge will be cast upon every 5 hours, or twice per ten hour shift.

With the average output of 80 squares per hour, pouring on barges of 20 tiers each, 20 barges will be required for continuous casting for five hours. At least two more barges must be in line to permit taking loaded barges out of line and replacing with empties in order not to interfere with continuity of mixer barge

operation. This computation illustrates conditions which govern length of line of casting barges.

Cement Barges are standard barges upon which has been placed a house of wood framing covered by galvanized corrugated iron. Each cement barge is equipped with two electrically driven belt conveyors as previously noted in the description of the mixer barge.

Shop Barges are standard barges with wood houses superimposed. One contains a combined carpenter and blacksmith shop for minor fleet and machine repairs; the other storage space for craft paper and empty cement sacks, as well as an electric sack shaker.

Spar and Header Barges—Standard barges are sparred out from the bank opposite every third casting barge to provide semi-permanent mooring for the line of casting barges. To provide sufficient "lead" for the up- and down-stream towing lines of the mixer barge, header barges must be placed at each end of the casting barge line. Old barges, incapable of bearing heavy loads and deep draft are thus utilized. Their number is further reduced by placing shop barges in the setup.

Spar barges are used to store distributed equipment and supplies, such as spare forms, paulins for covering mat in cold or wet weather, small No. 12 wires for ends of mat sections, and to support lights for night operations. The substitution for spar barges of a row of pile clusters is contemplated before the opening of next season.

Quarterboat—Most of the labor employed at the casting plant lives in the city of Greenville. However, a standard 2-story quarterboat is needed to quarter the personnel necessary for proper safeguarding of plant and property during emergencies and at night.

Towboats are 200-h. p. stern wheel boats. One is kept busy towing gravel and sand barges from digger to casting plant and is double-crewed. The other, single-crewed, performs necessary harbor work in shifting loaded barges from line to curing fleet, across the river, in replacing loads with empties, in spotting cement and gravel barges at the mixer barge when needed, etc.

The double-crewed boat also provides for emergencies during the night, principally in the event of fire or storm.

Sand and Gravel Barges are standard steel barges on which are placed boxes for the sand and gravel. The braces or knees which hold the sides of the boxes are steel; the siding is 4-in. lumber. The sand box and partition is 3 ft. high and approximately one half the size of the gravel box which is 1 ft. high. Greater height for the gravel box has been found unnecessary due to the natural piling of the gravel heaps. Each barge holds 200 cu. yds. of gravel and 100 cu. yds. of sand.

Gasboat Tenders, powered by 50-h. p. marine gasoline engines, are used to transport personnel and supplies; one for the curing fleet and one for the gravel digger.

The Gravel Digger is a 12-in. steam operated suction dredge which pumps the aggregates over ¼-in. grizzly bar screens. Two features are worthy of note: the upper discharge box and washing table, and the discharge chute and flipper pan over the gravel barge are rubber lined; an adjustable baffle plate under the lower end of the grizzly bars catches a portion of the sand washed through the bars and bypasses the sand through a flexible rubber elbow which connects with a 20-ft. section of light steel pipe. A light boom rig supports this pipe section and permits it to be swung horizontally so as to discharge on the lower end of the sand and gravel barge placed alongside the digger. In this manner both sand and gravel are simultaneously pumped on the same barge. The 16-ft. grizzly bars so completely de-water (and de-sand) the gravel that it is necessary to keep the gravel moving by pumping water from a small auxiliary pump to the head of the discharge chute.

The rubber lining was installed in October 1928 and is still in use. The only replacement required was a piece of 2 sq. ft. set in just below the lower end of the grizzly bars and at the head of the discharge chute. The rubber installation has been very satisfactory.

CASTING MAT SECTIONS

Steel mat forms are used (Fig. 2). Sliding clips which engage the wires hold the reinforcement in place in the center of the slab. The side pieces of the form are 3 x 2 x $\frac{3}{16}$-in. angles, the

cross members of 20 gage sheet steel shaped as required with welded seam along bottom of member. The end cross members are a special shape of $\frac{3}{16}$-in. steel. Wooden forms formerly used were discarded in favor of steel forms, since steel forms cost less in repairs, stripping and cleaning; hold reinforceing more accurately in place; are more accurately aligned and hold alignment

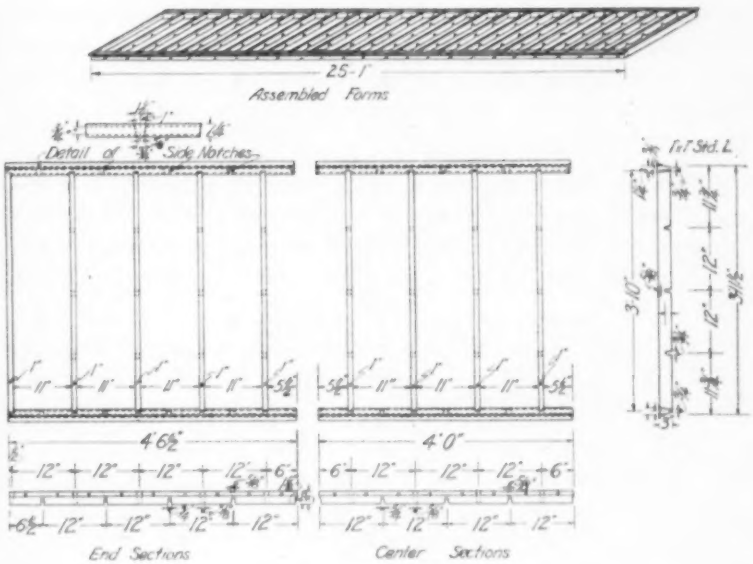


FIG. 2—CONCRETE MAT FORM

during pouring and finishing processes; require less time and labor to produce a satisfactory product.

SEQUENCE OF CASTING OPERATIONS

To supply a uniform production of mat sections to revetment field construction parties the casting barges must be so loaded as to permit approximately three completely loaded barges per day to be withdrawn from line, and replaced by a like number of empties. This requires, after a cessation of work, the skipping of certain barges in the line until a line shall have been established with barges in varying stages of completeness as to loading.

To give a proper picture of the cycle of casting operations it will be assumed that this loading echelon has been established. Then we will start at that stage of the cycle immediately following the completion of the previous pour upon any one unfinished barge. Favorable weather and temperature conditions are assumed.

Thirty minutes after pouring, two laborers go along the barge, disengage the reinforcement retaining clips in the forms, and carefully raise the forms about two inches letting them settle back to the extent caused by the dead weight of the form. This destroys practically any bond between block and form slightly before initial set.

Four hours after pouring the form cleaning gang lifts the forms and placing them on top of the mat section last made, proceeds to clean the forms. This cleaning is accomplished by lightly tapping and scraping the form section with small steel scraping tools.

This gang is closely followed by two small gangs which place reinforcement and craft paper. These materials are carried along the inside of line in small wood caulking flats.

Another gang follows up directly behind, lifts the form sections and reinforcement forward to the next tier, unrolls the craft paper separator, sets the reinforcement on the paper, places the form sections over the reinforcement, aligns the form sections and engages the reinforcement in the clips. The careful alignment of forms is very necessary to secure a straight line of the edges of the mat sections, else great difficulty will be occasioned field construction parties when the mats are assembled as later described.

The mixer barge, with its attached cement and gravel barges follows closely after. Several tiers in advance of the boom and bucket, two laborers spray the interior surfaces of the forms with oil. The mixer barge places an average of one square every 45 seconds. A general distribution of concrete along the form is effected by the bucket operator. Two laborers with broad hoes spread the concrete laterally to the edges of the forms. A gang promptly works and roughly smoothes the concrete with flat, square pointed, short handled shovels. A small gang with similar shovels follows carefully dressing the surface and insuring

a firm and complete working of the concrete into all parts of the form, and cleans surplus concrete from the deck of the barge.

The entire operation proceeds from the lower (downstream) end of the line. This direction is based on the handling of the mixer barge; it can be spotted more quickly and accurately when pulled against the current; it can be moved more rapidly downstream from one end of the line to the other, when the end of line is reached and the run must be recommenced. Fig. 16 shows casting layout and curing fleet moored across the river.

FORCES REQUIRED FOR OPERATION OF CASTING PLANT

SUPERVISION AND ADMINISTRATION
1 general foreman
1 assistant foreman
1 inspector (tests, etc.)

MISCELLANEOUS
7 clerks, property men, timekeepers.
13 laborers (cooks, flunkies, etc.)
10 white
13 colored

BANK GANG
1 foreman
1 crane engineman
25 laborers
2 white
25 colored

MAINTENANCE GANG
2 carpenters
1 welder
3 white

REINFORCEMENT GANG
1 foreman
4 laborers (cutters)
4 laborers (carriers)
2 laborers (placers)
1 white
10 colored

FORM CLEANING GANG
1 foreman
12 laborers (cleaners)
2 laborers (slippers)
1 white
14 colored

PAPER GANG
2 laborers (cutters)
1 laborer (carrier and placer)
3 colored

MIXER BARGE:
1 inspector
2 cranemen
1 Diesel engineer
1 electrician
1 winchman (mooring barge)
1 winchman (bucket)
1 laborer (bucket helper)
2 winchmen (mixers)
2 laborers (tramcars)
1 laborer (tramcar dumper)
2 laborers (batchers)
1 laborer (reservoir hoppers)
1 foreman (dressing gang)
20 laborers (dressing gang—rough)
6 laborers (dressing gang—finishers)
6 laborers (cement house)
12 white
37 colored

CURING FLEET
2 foremen
2 gasboat enginemen
10 laborers
4 white
10 colored

FLEET GANG
1 foreman
12 laborers

GRAVEL DIGGER:

1 steam engineer	4 deckhands
2 assistant steam engineers	2 mess and quarters
2 dredge runners	5 white
2 firemen	10 colored
2 oilers	

PLACING OF REINFORCED CONCRETE REVETMENT

General—The type of caving bank usually encountered presents a nearly perpendicular slope above low water line, and a more gradual slope below low water line, frequently approaching a 3 or 4 to 1 slope. Conditions of course vary, sometimes completely reversing the above-stated general condition.

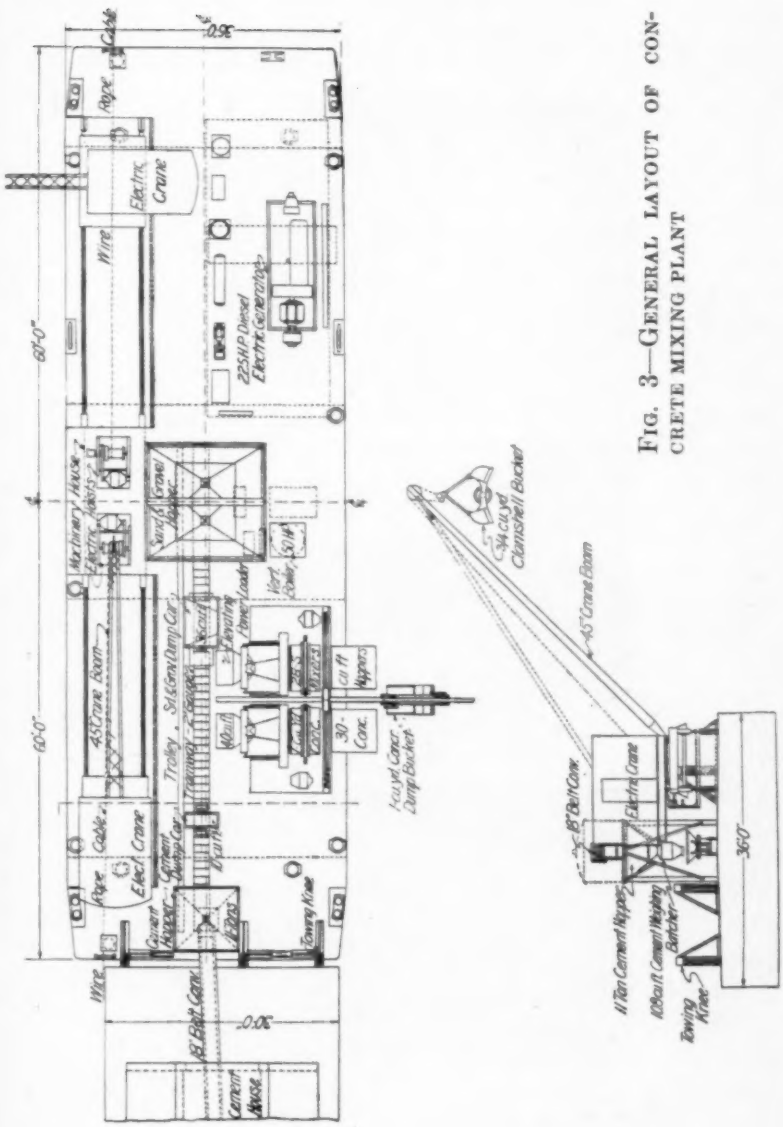
In approaching new revetment work the normal sequence of operations is

a. Clearing
b. Grading
c. Subaqueous mat
d. Upper bank paving.

Clearing—All trees and underbrush on the river bank are cut down and cleared for an average distance of 100 ft. landward from the upper edge to be established by the proposed grading. Brush and timber so cut down are carried landward beyond the line of the clearing and may not be placed in the river. A bank thus cleared facilitates grading operations and reduces likelihood of creation of snags should further caving result prior to placing of revetment.

Clearing is normally performed by hand; removal of stumps may sometimes be aided by the use of explosives. When clearing is heavy, tractors are of value. The character of the soil in the bank must be considered before explosives are used; in certain types of soil and under certain conditions of attack, the use of explosives may tend greatly to increase the instability of the bank.

Bank Grading—The upper bank grading is accomplished from the probable shore edge of the proposed subaqueous mattress to top of bank line on a 4:1 slope. All stumps and roots must be removed from the grade and the graded surface rendered reasonably smooth. Smoothing is accomplished, where soil conditions permit, with leaning wheel road grader and 5 ton caterpillar


Electric Crane
Wire
225 HP Diesel Electric Generator
Machinery House
Electric Hoists
Sand & Gravel Bunker
Hot Water Tank
45' Crane Boom
Trolley
Sand & Gravel Dump Car
Concrete Mixer
Gasoline Power Unit
Water
Conc.
30' Conc.
Cut-off Hoppers
Hoyd Concr. Dump Bucket
Cable
Pipe
Elect. Crane
Cement Dump Car
Earth
Cement House
Conveyor
Conveyor
Conveyor

tractor. In heavy soils and short stretches of repair work saving is effected by hand shovel dressing. The river stage at time of

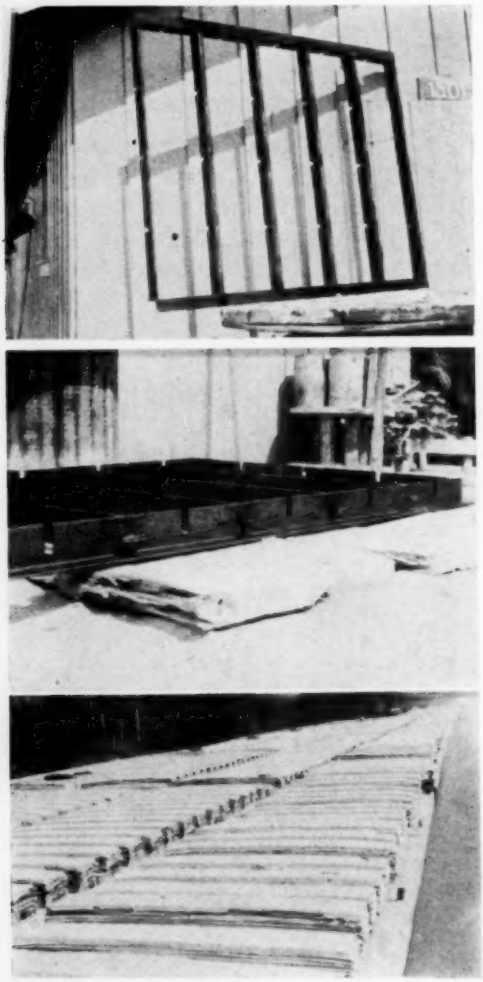


FIG. 4—STEEL FORM SECTION

FIG. 5—SECTION OF STEEL FORM FOR ARTICULATED REINFORCED CONCRETE MAT. FIVE OF THESE SECTIONS ARE NEEDED FOR CASTING EACH SECTION OF "SQUARE"

FIG. 6—LOADED MAT BARGES

grading governs extent to which lower edge of upper bank grading may be carried. It is desirable that it extend down to low water line. Explosives may be utilized to assist in grading, but care must be employed in their use.

Underwater slopes should not be steeper that 1 to 1; indeed it is difficult to hold successfully any slope steeper than 3 to 1. Where steep underwater slopes exist, it is sometimes advisable to sink only the subaqueous mat. Subsequent action of river currents may then cause further recession of bank, but the subaqueous mat, though partially lost, remains as a heel to establish the foot of the slope which will gradually, by placing of later successive mats, assume a slope which can be held successfully.

Water pocketed upon the bank in old levee borrow pits, ditches, or depressions may, when overbank river stages recede, seep through underlying strata of bank and accelerate caving. Accordingly drainage ditches are constructed into the river from all depressions within 300 feet of the top of bank line, and frequently from farther distant pools. These ditches must be paved for a short distance at their riverside ends to prevent scours from draining waters; this ditch paving must be firmly connected with upper bank paving to prevent failure along connection.

Considerable trouble results from underground seepage from lakebeds lying to the landside of controlling levees. Proper drainage from such sources is much more difficult than in the case noted above concerning riverside pools; special study is necessary for each locality, and in some instances the problem is as yet without solution.

Bank Grading Operations—Bank grading is usually performed by hydraulic graders. These hydraulic graders in the Vicksburg district consist of a barge on which is mounted two multiple stage centrifugal pumps, driven by steam turbines, which pump river water at 250 lbs. per sq. in. pressure through 5-in. pipe lines to 1¼-in nozzles (1⅛-in. for sandy bank). These nozzles operating on the bank are utilized to undercut the bank material and slope off the bank to the desired 4 to 1 slope. The dual unit (boiler, turbine, and pump) exists on each grader. Each pump operates one nozzle. Four-inch rubber sections provide a flexible connection between pipe and nozzle. All pipe sections are connected by flexible Moran type ball and socket joints. The second

story of the cabin erected on each grader provides living and messing facilities for the necessary operating personnel. Three of these hydraulic graders operate in the Vicksburg Engineer District.

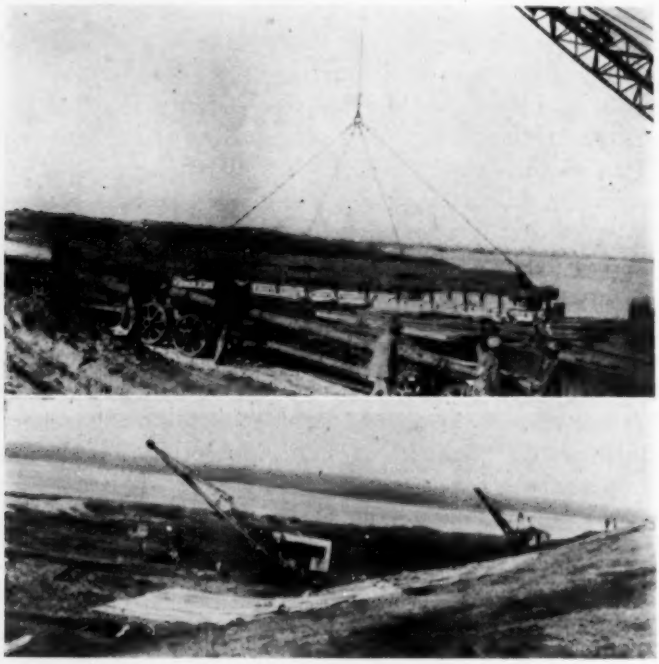
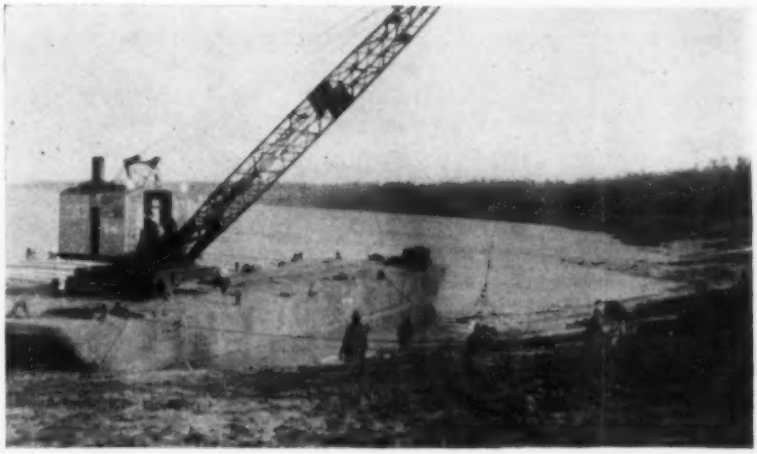


FIG. 7—PAVING SCOURED LEVEE WITH MAT SECTIONS. HAULING BY TRACTOR AND WAGON
FIG. 8—PAVING SCOURED LEVEE WITH MAT SECTIONS

An experimental mechanical bank grader was erected and operated in the Vicksburg Engineer district during the 1929 season. This mechanical grader consists of a revolving, levee-building steam dragline mounted on two standard steel barges placed side by side and firmly connected. The machine was originally constructed to handle a 5 cu. yd. bucket on a 165 ft. boom. The boom has been shortened to 150 ft. and reinforced. Instead of the dragline bucket, earth is moved by a curved scraper placed at the end of two dipper sticks. The dipper sticks remain

in a nearly vertical position during the earth moving operation, sliding up and down in guides on a traveling cradle which moves in and out along the lower members of the boom. A dragline



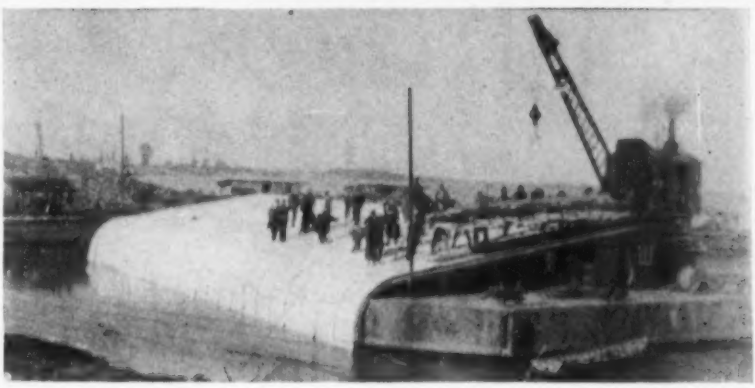


FIG. 9—PAVING UPPER BANK WITH FLOATING LOCOMOTIVE CRANE
FIG. 10—ASSEMBLING MAT SECTIONS, SINKING BARGE (25 SQUARE LAUNCH)

pulls the scraper-head toward the machine; a backhaul cable pulls it toward the top of the bank; two lines govern the travel of the cradle along the boom, while a fifth line regulates the elevation of the scraper-head and slides the attached dipper

sticks through the guides on the cradle. The entire machine may be revolved on its turntable or self-propelled on its rail track along the barges. Two operators are required to manipulate the numerous controls.

The machine operates with the long dimension of its supporting barges parallel to the bank line. Mooring barges, headed into the

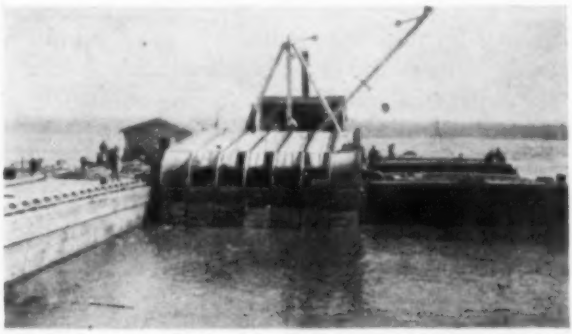
FIG. 11—PAVING UPPER BANK SHOWING MAT-LIFTING FRAME AND GRAPPLE HOOKS

FIG. 12—PAVING UPPER BANK WITH DERRICK BOAT

bank, are placed against the supporting barges, above and below. The machine and its supporting barges may thus be moved to and from the bank between the mooring barges.

This machine grades 12 ft. below water surface. Following preliminary experimental operation at the beginning of the season, and with acquisition of experience in handling by oper-

ators, the machine can well be considered a success. The absence of water renders its use desirable on sloughing banks; its costs during the latter part of the season were lower than those of hydraulic graders; it accomplishes underwater grading; on repair work on short stretches of bank it is much more economical since its mobilization and demobilization is far simpler.



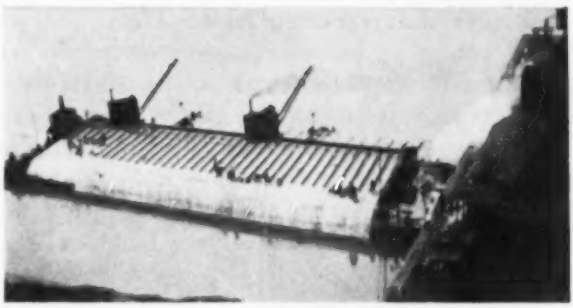


FIG. 13—END CONNECTING MAT BOAT
FIG. 14—LARGE MAT SINKING BARGE (35 SQUARE LAUNCH)

It has the disadvantage of much higher first cost. Contemplated improvements in future design embrace lowering of entire machine nearer to deck of supporting barges; lowering of dragline sheave; installation of mechanical devices for moving in and out along mooring barges instead of present manual operation; utilization of Diesel or Diesel-electric power.

Subaqueous Mat—Subaqueous articulated type reinforced

concrete revetment used in the Vicksburg Engineer district consists of standard articulated mat sections having their outside longitudinal reinforcing wires securely fastened to a standard steel anchoring cable. This fastening consists of clips (U-bolts, bridge and two nuts, all with japanned finish) set at intervals not greater than 5 ft.; each clip encloses in its grip the outside longitudinal reinforcing wires of adjacent mat sections together with the included anchoring cable.

An additional fastening, including the same reinforcing wires and anchoring cable as the clips, is placed at one foot intervals between the clips; this additional fastening consists of a galvanized wire of No. 12 gage, doubled and tightly twisted.

Anchoring cable consists of 7/16 in. or 1/2 in. standard galvanized steel cable having breaking strengths, respectively, of 9,000 to 10,000 lbs. and 14,000 to 15,000 lbs. The size of anchoring cable used depends on depth of river, steepness of underwater slope, and velocity of current in the locality of construction. Normally upstream five cables of each mat are 1/2 in. the remainder 7/16 in.

Each anchoring cable is securely fastened to an anchorage consisting of a deadman, sound stump, or other secure anchorage. These anchorages exist or are placed at least 10 ft. inshore from the upper edge of the graded bank, and must develop a resistance not less than twice the combined ultimate strength of attached anchoring cables. Anchoring cables may not deviate more than 2 ft. from a straight line from the mat fastening to the anchorage. Not more than two adjacent cables are fastened to any one anchorage.

Subaqueous mat is assembled prior to launching. The mat sections are placed with 25 ft. dimensions approximately perpendicular to top of bank line. Any number of mat sections up to capacity of the mat placing barge, may assemble adjacent to each other; a portion of mat so assembled is termed a "launch." Successive launchings must be continuous from waters-edge to deepest water; this continuity is secured by the clip fastenings to anchoring cables and by twisting tightly together the wire loops in the end blocks of contiguous and abutting mat sections of successive launchings.

A continuous series of successive "launches" from waters-edge

to deepest water is called a "mat." Work is normally begun at downstream end of the proposed revetment. Successive mats are overlapped a minimum of ten feet, the upstream mat overlapping the next downstream mat. Overturning of upper edges of mat by the current is thus prevented.

Upper Bank Paving consists of that portion of the revetment extending from top-of-paving-line to the upper edge of the subaqueous mat. The top-of-paving-line is an elevation or contour representing the average top-of-bank-line or limiting line designated for protection in a given locality; it is usually expressed in terms of an elevation on the nearest United States Engineer Department river gage.

The upper bank paving may consist of:

a. Monolithic reinforced concrete.

b. Standard reinforced concrete mat sections.

c. Concrete blocks.

d. Riprap stone.

The last two named are laid by hand. Only the first two were used in the Vicksburg District during the 1929 season; the following descriptions will be limited to these first two types.

Monolithic concrete paving—Monolithic upper bank paving consists of a sheet of reinforced concrete with minimum thickness of 4 in.; 2000 lb. concrete is used. A reinforcing mesh is used similar to that for mat sections.

The concrete is brought to a reasonably smooth surface. The reinforcement is finally imbedded, and may not be closer than ½ in. to the upper surface of the concrete. Adjacent sections of reinforcement are overlapped at least 6 in. Expansion joints are provided. Monolithic paving is not placed at temperatures below 40° F.

Anchoring cables of the subaqueous mat are firmly imbedded in the monolithic paving.

An anchoring curb is constructed along the top-of-paving-line by digging a trench whose minimum dimensions are 4 in. wide and 18 in. deep. The reinforcing mesh of the paving is turned

down into this trench and both paving and curb are poured simultaneously.

Paving with mat sections—As nearly as practicable, bank paving with mat sections is merely a continuation of the subaqueous mat up to the top-of-paving-line. Wherever possible, anchorage cables of the subaqueous mat are included in the fastenings of the sections of the upper bank paving; except where it is thus possible to include anchoring cables, clips are omitted and all fastenings are made of twisted wires only. The 25 ft. dimension is always placed perpendicular to the top-of-paving-line.

A variation extensively used this past season consists of laying upper bank mat sections from waters-edge up, overlapping by two blocks the upper section over the next lower, rather than butting the ends. This method gives a stronger job and obviates construction difficulties occasioned by attempting to fit ends of sections where the bank contours are irregular.

When it becomes necessary to overlap adjacent mat sections laterally, the upstream section overlaps the downstream square.

CONNECTIONS

Connecting Mats—The most desirable condition for construction of revetment exists where it is possible to complete grading at a low river stage, place subaqueous mat at nearly bankfull stage, and complete upper bank paving at a medium stage. Such ideal conditions rarely follow in sequence. It is unfortunately oftener necessary to place subaqueous work at a low stage and complete upper bank paving at a higher stage.

When this latter condition obtains, it is necessary to construct a "connection" from the subaqueous mat to an elevation slightly above waters-edge. Such connection consists of a "connecting mat" which overlaps the subaqueous mat at least 10 ft. and not more than 15 ft.

Connection between monolithic upper bank paving and subaqueous or connecting mat is performed, in the dry, by extending the reinforcement of the monolithic paving down over the lower mat sections for an overlap of 6 ft. Standard monolithic concrete is then poured to a thickness of 2 in. above the mat sections of the lower mat. The consistency of this concrete and the work-

manship involved in its placing are such as thoroughly to imbed the anchoring cables, exterior reinforcing wires and interstices of the mat sections, as well as the overlapping reinforcement of the upper bank paving. The concrete of the connection is poured practically simultaneously with and thoroughly bonded to the concrete of the upper bank paving.

PLANT AND EQUIPMENT

The following special equipment is used in concrete revetment construction in the Vicksburg District.

Mat placing barges
Upper and lower mooring barges
End connecting mat barge
Tower paving barge
Locomotive crane barges
Derrick barges

A brief description of each special type follows:

Mat Placing Barges—During the 1929 season two mat placing barges were used. The older and larger of the two permits the construction of a 35-sq. launch; this barge is steam powered. The squares are transferred from a loaded casting barge lashed to the outside of the mat placing barge by two locomotive cranes to the sloping deck of the placing barge. Each mat section rests on two parallel rows of rollers. Anchoring cables run from reels placed under the construction ways and are led back and interlaced through two friction pulleys, through the upper deck, down between adjacent mat sections and over the inside curved edge of the placing barge. A steam winch drives the pay-off mechanism through a long shaft and train of gears. The gear train is so arranged that mat may be paid off in any one or combination of five 7 square sections, or as a 35 square unit. Steam capstans on each end of the placing barge provide power for moving the placing barge along the lower mooring barges.

The smaller placing barge provides for a launch of 25 squares. A single steam locomotive crane is installed. All other power is secured from individual gasoline-engined units. The inner (shoreward) side is constructed with a longer overhanging rake, which places upper edge of mat nearer to waters-edge than does large placing barge.

Both of the above barges are of steel construction throughout.

Lower Mooring Barges—These barges (360 ft. total length) are of steel for the large plant and of wood for the smaller. Along the downstream edge there is a track along which a heavy trolley runs. The placing barge is lashed to this trolley.

Six steam winches placed at intervals along the lower mooring barges furnish power for drawing the mooring barges and attached placing barge upstream for sinking successive mats. Each winch is equipped with 1200 ft. of steel cable which is led out through a "rooster-head" fairlead to fastenings on the upper mooring barges.

Upper Mooring Barges—The upper mooring barges (240 ft. total length) serve as a spread anchorage for the cables from the lower mooring barges, and are in turn fastened to shore anchorages above. They are of wood for the small plant, and of steel for the large plant.

End Connecting Mat Barge—In order to place connecting mats with upper edge above the waters-edge, the mat launches are paid off over the end of the connecting mat barge whose rake, for the first launch, overhangs the bank. The present barge is of wood, and provides for a launch of 7 squares. A stiffleg derrick transfers mat sections to the construction platform.

Tower Paving Barge—This barge (Fig. 15) is of special size and of steel construction. An Insley Tower, with chute and counter-weighted distributing trussed chute distributes concrete for monolithic bank paving. A skiphoist conveys the concrete to the chute from a 1-yd. mixer. Sand and gravel hopper is equipped with weighing batchers. Cement is measured by the sack, and is conveyed to the mixer by ladder conveyor from a cement storage house. A stiffleg derrick loads sand and gravel from an attached barge to the hopper. Steam power is utilized throughout.

The maximum output attained by this plant during the past season (and during its use) was 275 squares of standard monolithic paving in ten hours. Its average output is 200 squares per day.

Locomotive Crane Barges—Locomotive crane barges consist of a steam railway locomotive crane with 65-ft. boom on a standard sized steel barge. Special interior bracing is built into the barge. These units are utilized in placing mat sections on

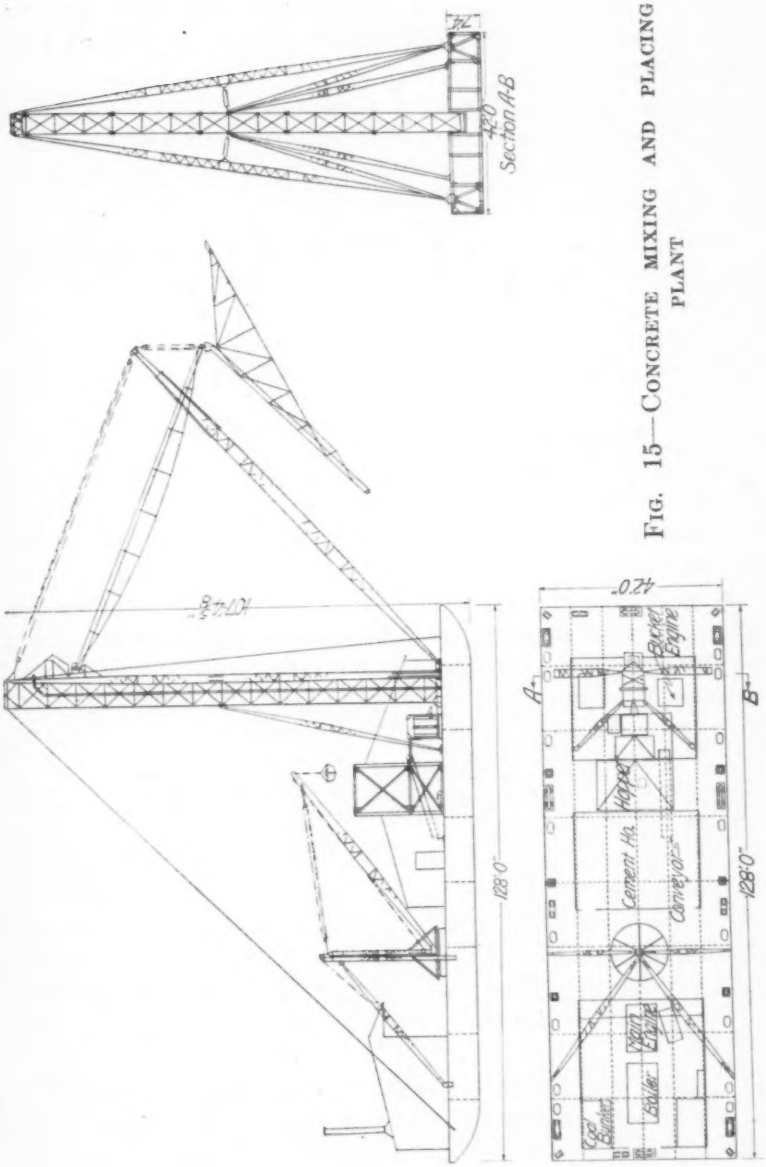


FIG. 15—CONCRETE MIXING AND PLACING PLANT

the bank, both for connections where upper edge of subaqueous mat is barely under water surface and for paving upper bank with mat sections.

The locomotive crane barge possesse advantages of speed and flexibility of operation and the disadvantage of too short reach. During low water stages the present equipment cannot reach top-of-paving-line with mat sections.

Derrick Barges—A derrick barge consists of a stiffleg derrick mounted on a standard barge. Boom lengths up to 90 ft. are available in the Vicksburg district. These derricks are slower and of less flexibility of operation than the locomotive crane barges, but possess the advantage of longer reach.

SEQUENCE OF OPERATIONS SUBAQUEOUS MAT

Following clearing and grading of bank, the next operation in revetment construction is the placing of subaqueous mat. The following description applies to construction with the large placing barge.

A bank gang of a foreman and 15 to 25 laborers begins the construction of anchorages; digging deadman holes, procuring logs for deadmen, and securing anchoring cables to anchorages. Short pieces of anchoring cable, sufficient to make the fastening at the anchorage and reach slightly beyond the waters-edge, are secured to the anchorages and led down to the waters-edge. The anchoring cables from the mat boat are later tied and clipped to these cables.

Meanwhile the inspector has set up his instrument on the local revetment base line and spotted the alignment of the downstream edge of the lower mooring barges. The upper mooring barges are placed approximately 1200 ft. upstream from the starting point, the lower mooring barges and placing barge drop downstream to proper starting point, and a loaded casting barge placed outside the placing barge.

One launch of mat already hangs over the edge of the placing barge. Why it is there will later be explained. The placing barge is drawn in as close to the bank as possible. Two unloading gangs of 6 laborers each take position at the upstream end and middle of loaded casting barge and engage the hooks of the grappler

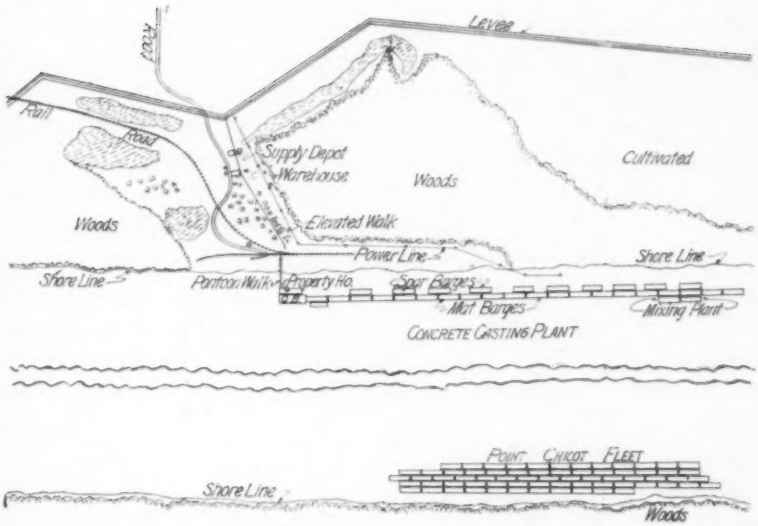


FIG. 16—LAYOUT OF CASTING PLANT; CURING FLEET MOORED ACROSS THE RIVER

frame suspended from the booms of each of the two locomotive cranes. These grappler hooks engage the exterior portion of each of the transverse reinforcing wires.

The locomotive cranes transfer the mat sections to the construction ways of the placing barge. The cranes begin placing sections at the upstream end and middle of the construction ways, and continue the transferring operation successively toward the downstream end.

Two gangs, one following each crane, place and unhook the mat sections from the grappler frames, and assemble the launch by fastening clips and twist wires. Each gang consists of a foreman, 6 laborers (placing and unhooking), 5 laborers (fastening clips), 12 laborers (twisting wires).

When the launch is completely assembled the payoff mechanism is set in motion and the placing barge also moved out along the mooring barges at a rate commensurate to the slope of the underwater bank and the payoff of the mat. Rate of mat payoff is two feet per minute.

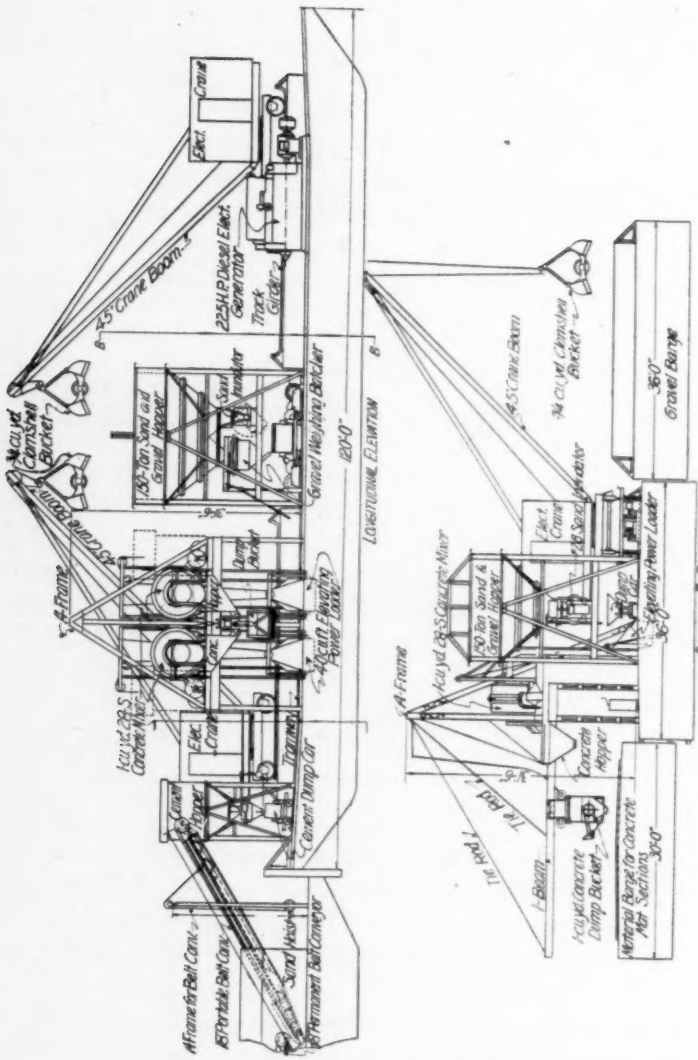


FIG. 17—ELEVATION AND SECTION, CONCRETE MIXING PLANT

Previous soundings have determined the underwater bank slope; a scale of feet painted along the trolley rail provides a measure for the position of the placing barge at the conclusion of each launch.

Successive launches are constructed in like manner, the mat being paid off in the manner of a huge roller-topped desk upon the completion of assembly of each launch.

The inspector has previously made a sounding from the mooring barges at the point where the sinking plant will rest when cables are cut off after the last launch. This sounding is furnished to the overseer foreman in charge of the placing barge in order that the cables may be cut at the proper length.

When the last launch of the mat has been assembled, payoff is resumed and continued until all but 25 ft. of the required cable length has been paid out. This leaves the last launch suspended under water, all the rest of the mat resting on the bank. Payoff is stopped and the first launch for the next mat assembled.

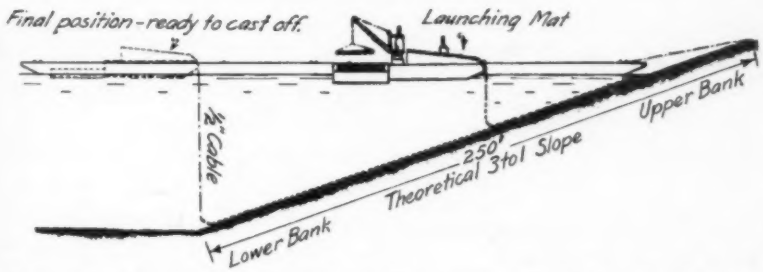


FIG. 18—DIAGRAM ILLUSTRATING SUCCESSIVE POSITIONS OF MAT BOAT DURING SINKING OPERATIONS

When this launch is assembled payoff of last launch is completed, leaving the first launch for the next mat hanging over the landward edge of the placing barge. A similar situation was described as a setup of the placing barge when the present mat was begun. Five laborers in skiffs and equipped with bolt cutters then move along the suspended anchoring cables and cut them at the water surface, completing the first mat.

The placing barge is them moved to the landward end of the mooring barges to reduce strain on cables, and the lower mooring

barges moved upstream 130 ft., aligned by the inspector, and the process repeated. After six to eight mats have been placed, it is necessary to move the upper mooring barges upstream. To do this the entire plant must be moved up by harbor boat and the lower mooring barges and placing barge dropped down again into position. Such a move requires three to five hours.

ORGANIZATION FOR CONSTRUCTION

During 1929, 3 field construction parties operated in the Vicksburg District. Only major items of equipment are listed:

a. Revetment party No. 1.
Extensions—New work
Large placing unit
2 quarterboats
1 harbor tug.

b. Revetment party No. 2.
Major repairs—minor
Extensions—concrete mat paving
Small placing unit
1 quarterboat
2 locomotive crane barges
1 harbor tug.

c. Revetment party No. 3.
Monolithic paving
Connections—minor repairs
Tower paving unit
End connecting mat barge
1 quarterboat
1 derrick boat
1 harbor towboat.

CONSTRUCTION OUTPUT

The following tabulation indicates maximum and average performances per ten hour day for revetment construction units. Maximum output results from a combination of favorable circumstances; averages are based on mean output per working day.

Unit	*Maximum*	*Average*
	Sq.	*Sq.*
Large Placing Barge	750	650
Small Placing Barge	500	400
End Connecting Mat Barge	200	150
Tower Paver	275	200
Locomotive Crane Barge	300	250
Derrick Barge	225	175

FACTORS GOVERNING DESIGN OF CONCRETE MAT

The mixer barge was originally designed for an output of one square per minute. By study of its actual operation and resulting minor changes, it has been possible to eliminate all "bottle necks" within its cycle of operations, and to move this "bottle-neck" to the final operation, i. e., the capacity of the dressing crew.

An experimental run for maximum capacity over a one hour period netted 105 squares! This is slightly over 34 seconds per square, or a cycle per mixer of 68 seconds: 7 seconds charging, 50 seconds mix, and 11 seconds discharging. Needless to add such a pace could not have been maintained for even 5 hours by the dressing gang.

Moreover to give a five hour period between runs, thirty barges would be needed in line, more time would be lost moving between ends at completion of run, material placing haul would be lengthened, and numerous other disadvantages would result.

Accordingly an average output of 80 squares per hour has been deemed an efficient mean. This rate will supply construction needs now apparent, and increased demands can more efficiently be met by increasing number of hours or shifts per day.

Output is determined by a balance between needs of construction parties, length of time available for curing, and number of barges available.

It has been found that the greatest stress placed upon the mat sections is set up during the process of assembling and sinking. The mat sections are dropped through a distance of approximately 12 ins. when placed upon the mat sinking barge. They rest on this barge upon rows of rollers set 40 ins. on centers. It was found by a series of tests that mat sections whose individual blocks could sustain a concentrated center of load of 500 lbs. when supported on 40 in. span could successfully be placed on the riverbank with less than 2% crackage. Hence this method of testing and measure of strength was adopted rather than compressive cylinder tests.

With this in mind, mixes are designed to produce this strength within the time available between casting and sinking. When weather conditions are propitious and ample curing time and

barges available, mixes as lean as 11 gal. per sack have been successfully used. With 14 day cure water-cement ratio of 9 gal. per sack on a 1:2.5:4.3 have fulfilled requirements.

After the concrete is once placed on the riverbank, its stresses diminish while its strength increases. Its life is mainly determined by the resistance of its exterior reinforcing wires and anchoring cables to corrosion. When new, the reinforcing will develop much more strength than the concrete; the size of the reinforcing is principally to increase its life against corrosive action.

Maximum corrosive action occurs above low water, over that portion of revetment most subjected to alternating periods of action of air and river water. This range is usually between river stages varying from 5 ft. to 25 ft. on nearest gage. For this reason 6,000 mat sections with copperweld reinforcement were made and placed between these ranges at several points in the Vicksburg District during the past season. The success of this type of reinforcement is subject to the results of future observations. The copperweld reinforcement furnished during the past season presented difficulties in casting and handling mat sections because of undue softness and resultant binding of reinforcement. It is believed that this disadvantage can be corrected.

Readers are referred to the JOURNAL *for December 1930 (Proceedings, Vol. 27) for discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1930.*

Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams

BY WILLIS A. SLATER* AND INGE LYSE†

INTRODUCTION

1. *General Statement.* The practice in designing reinforced concrete beams in the United States has been shaped and controlled largely by the recommendations of a Joint Committee on Concrete and Reinforced Concrete appointed in 1903, which made its final report in 1916. This Committee recommended the straight line formula for use in computation of working stresses in reinforced concrete beams, that is, a formula which assumes a constant modulus of elasticity. The working stress so computed was fixed at a maximum of 32.5 per cent of the strength of the concrete as determined from compression tests of 8 by 16-in. cylinders 28 days old. The recommended working stress in the reinforcement was 16,000 lb. per sq. in. This combination of stresses requires about 0.75 per cent reinforcement.

This recommendation is found in the 1909 and 1912 progress reports and is repeated in the final report of 1916. It appears to provide a factor of safety of approximately 3 for compressive stresses and 2.5 for tensile stresses. Test beams proportioned to meet these conditions have always failed in tension in the reinforcement with a factor of safety between 2.5 and 3.0‡ However, tests on beams with sufficient reinforcement to develop compression failure have shown factors of safety ranging approximately from 4 to 8, but with the majority of cases above 5 when judged on the basis of working stress in the concrete of 32.5 per cent of the cylinder strength.

Such conservative stresses were proper for the design of earlier days and in many cases may still be proper. However, two

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‡Westergaard and Slater, "Moments and Stresses in Slabs." *Proceedings*, American Concrete Institute, Vol. 17, page 415, 1921.

important results have followed the adherence to these working stresses computed from the straight line formula. Many designers appear to have lost sight of the facts that (a) the compressive stresses in reinforced concrete beams computed by the straight line formula for loads near the maximum generally are considerably greater than the compressive strength of the control cylinders, and (b) that it is sometimes desirable and permissible to use a larger percentage of reinforcement than that which will give a computed stress in the concrete of 32.5 per cent of the cylinder strength, simultaneously with 16,000 lb. per sq. in. in the steel.

With the improvement gained in recent years in our knowledge of the properties of concrete and control of its quality, it is desirable to make modification in the relation between the computed compressive stresses and the computed tensile stresses, which will result in utilizing both materials (concrete and steel) economically. The fact that among the thousands of beams tested in laboratory investigations only a very small percentage have failed in compression indicates that in general more steel is required to utilize the strength of the concrete sufficiently.

In the 1921 and the 1924 reports of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete the maximum working stress for concrete in flexure was raised to 40 percent of the cylinder strength. This working stress, however, has not been universally accepted.

This investigation was planned to determine the relation between the compressive strength of 6 by 12-in. control cylinders and the strength of the same concrete computed from beam tests in which the concrete failed in compression. The investigation included two series of tests, one in which with beams of equal dimensions the cylinder strength of the concrete ranged from about 1400 to 5800 lb. per sq. in. and another series in which for two different strengths of concrete (2800 and 4000 lb. per sq. in.) the total depth of the beams varied from 6 to 17 inches. A large enough percentage of reinforcement was used in all cases to insure that failure would not occur in tension of the longitudinal reinforcement. Web reinforcement was used to guard against tension failure of the web. Thus it was intended to eliminate all causes of failure except compression.

This investigation was carried out in the Fritz Engineering Laboratory of Lehigh University, Bethlehem, Pennsylvania, from January to March 1930.

2. *Acknowledgment.* The cement for the investigation was furnished by the Lehigh Portland Cement Company of Allentown, Pennsylvania, the reinforcement steel by the Rail Steel Bar Association of Chicago, Illinois and the aggregates by the Warner Company of Philadelphia. All of these materials were furnished without charge. All other expenses were borne by the Portland Cement Association of Chicago and Lehigh University.

The success of this investigation has been largely due to the interested cooperation of the men connected with the work. Especial acknowledgment is made to C. C. Keyser, Laboratory Assistant and C. L. Kreidler and R. E. Gohl, Senior students in the Department of Civil Engineering.

3. *Earlier Tests.* The earliest study the writers have found comparing the strength developed in the beam with that developed in the cylinder was reported by Dr. Fritz Emperger in "*Beton und Eisen,*" Heft I, 1903, page 23. This study was based upon tests carried out at different places, some by Ransome in Boston and some by Sanders in Amsterdam, Holland. Values of the ratio of strength of concrete computed from the beams tested by Sanders to the strength shown by the control cubes are shown in Fig. 1. The strengths for the beam tests were computed by a straight line formula. As a result of this study the "German Committee for Reinforced Concrete" initiated a more extensive investigation of the same subject. The tests by the German Committee were carried out at the Technischen Hochschule at Stuttgart, Germany, under the direction of C. Bach and O. Graf. The results of these tests were originally reported in Heft 19, "*Deutscher Ausschuss für Eisenbeton*" and have been abstracted from Mörsch's book "Der Eisenbetonbau," Fifth Edition, page 330, and plotted in Fig. 1. The control specimens for these beam tests were 30-centimeter (11.8-in.) cubes. In order to compare the results on the basis of ordinary 6 by 12-in. cylinders the corresponding ratios of beam strength to cylinder strength are shown at the right-hand side of the diagram. For convenience this ratio is termed the beam-cylinder strength ratio. In making

the conversion the strength per unit of area of the cube has been taken as 13 per cent* greater than that for the 6 by 12-in. cylinder.

A similar study of results from different sources in the United States was published in a paper by Slater and Zipprodt in the

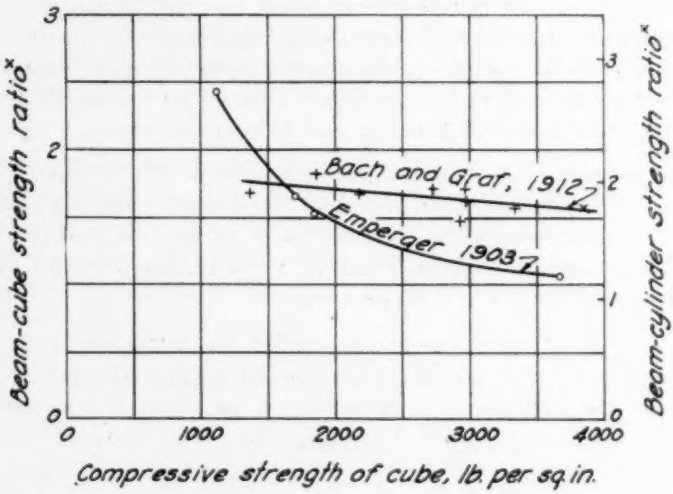

FIG. 1—FOREIGN TESTS OF HEAVILY REINFORCED CONCRETE BEAMS

Proceedings of the American Concrete Institute, 1920, page 120. The essential results from these tests are shown in Fig. 2.

Figures 1 and 2 show that for all these tests the strength of the concrete computed by the straight line formula was considerably greater than the strength of the concrete as shown by tests of cubes or cylinders, that is, the beam-cylinder strength ratio was always greater than one. The tests also indicate that the beam-cylinder strength ratio increased as the strength of the concrete decreased. The variation of this ratio with the strength of the concrete was less marked for the tests reported by Bach and Graf than for the others reported.

4. *Program.* Since the study was mainly on the compression

*H. F. Gonnerman: "Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete," *Bulletin 16*, Structural Materials Research Laboratory, 1925.

resistance of the beam, the most important consideration in the layout of the program was to design the beams in such a way that failure would occur due to crushing of the concrete in the beam. In the calculations the assumptions made were that the yield-point stress would be 40,000 lb. per sq. in. for the reinforcement, and that a large enough amount of reinforcement would be

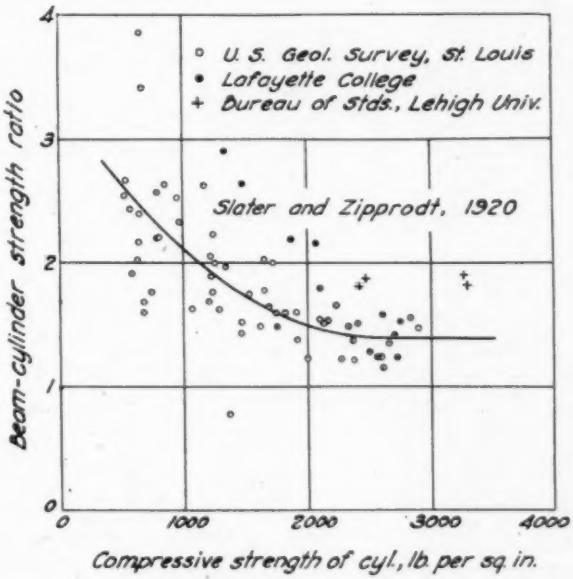

FIG. 2—TESTS OF HEAVILY REINFORCED CONCRETE BEAMS MADE IN UNITED STATES

used to avoid danger of failure by tension in the steel. Additional security against tension failure resulted from the fact that the rail steel reinforcement supplied for the tests had a yield-point of approximately 65,000 instead of 40,000 lb. per sq. in.

For the series in which the compressive strength of the concrete varied the total depth of the beams was 13 in. The depth from the compression surface of the concrete to the center of gravity of the steel, here termed the effective depth, was 10 in. The designed cylinder strengths of the concrete for this series varied from 1000 to 5000 lb. per sq. in. and are shown in Table 1.

TABLE 1—DESIGN OF REINFORCED CONCRETE BEAMS

Group No.	Designed Concrete Strength Lb. per sq. in.	Ratio of Reinforcement		Reinforcement Bars			Dimensions		Computed Shearing Stress Lb. Per Sq. In.
		Design	Actual	No.	Size	Area	Depth to Steel In.	Width In.	
1	1000	.02	.021	4	¾"	1.77	10.2	8.2	222
2	2000	.03	.028	1 4	⅞" ¾"	2.37	10.3	8.2	333
3	3000	.04	.037	4	1"	3.14	10.3	8.2	444
4	4000	.05	.047	5	1"	3.92	10.1	8.2	555
5	5000	.06	.056	6	1"	4.71	10.2	8.3	666
6	2000	.03	.030	1 5	¾" 1"	3.45	14.2	8.2	467
6A	3000	.04	.039	1 5	¾" 1"	4.53	14.1	8.2	680
7	2000	.03	.028	1 4	⅞" ⅞"	2.85	12.2	8.3	400
8	2000	.03	.031	1 2	1" ⅞"	1.99	8.0	8.1	267
9	2000	.03	.032	1 2	⅞" ¾"	1.48	5.9	7.9	200
10	2000	.03	.030	1 2	⅞" ¾"	.99	4.1	8.0	133
10A	3000	.04	.040	1 3	⅞" ¾"	1.33	4.1	8.0	200

Beams in Group 1 to 7, inclusive, have ¾-in. U stirrups spaced 3 in. center to center except No. 5 and 6A which have ⅝-in. double U stirrups 3 in. on center. Beams 8 to 10A, inclusive, have ¾-in. bars at an inclination of 20 degrees with longitudinal reinforcement and spaced 9 in. center to center.

In order to study the effect of variation in depth of the beams on the beam-cylinder strength ratio a series of tests was planned in which the effective depth of the beams varied from 4 to 14 in. as shown in Table 1. Two strengths of concrete (designed as 2000 and 3000 lb. per sq. in. cylinder strength) were used with each of these variations in depths of beams.

The computed shearing stress for the loads calculated to cause compression failure in all beams are also given in Table 1. Obviously most of them are so high as to require web reinforcement to prevent diagonal tension failure. For beams having a depth to center of gravity of steel of 10 inches or more, vertical stirrups were used. For shallower beams the depth was too small for proper anchorage of vertical stirrups and inclined web reinforcing bars were used. As a criterion for the minimum amount of web reinforcement required the formula

$$v = (.005 + r) \cdot f_v$$

was used. In this formula v is equal to the shearing stress, r is equal to the ratio of web reinforcement and f_v is equal to tensile stress in reinforcement here taken as 40,000 lb. per sq. in. This formula was derived from tests carried out for the United States

Shipping Board during the war.* However, the minimum size of the stirrups was fixed at $\frac{3}{8}$ in. diameter and the maximum spacing at 3 in. for the vertical stirrups. This generally resulted in considerably more web reinforcement than that called for in

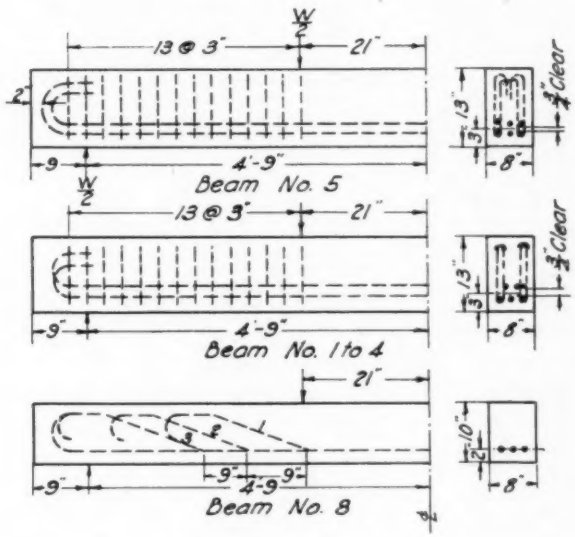

FIG. 3—DESIGN OF REINFORCED CONCRETE BEAMS. NOTE: ALL WEB REINFORCEMENT IS $\frac{3}{8}$" DIA.

the formula. Fig. 3 shows the arrangement of the web reinforcement for all the beams.

All the beams were 8 in. wide and 11 ft. long.

Three beams of each kind were made for each group. The control specimens generally consisted of 6 by 12-in. cylinders and three cylinders were made with each beam. Prisms 8 by 8 by 12 in. were made with two of the beams for testing under eccentric load. At a later date three prisms and three cylinders were made in order to secure further information on the properties of the concrete.

*Slater, Lord, and Zipprodt, "Shear Tests of Reinforced Concrete Beams," Technologic Paper No. 314; National Bureau of Standards, 1926.

MATERIALS

5. *Cement and Aggregates.* The cement and aggregates used in this investigation came from the same lots as those furnished for a series of column tests which is being carried out at Fritz Laboratory for the American Concrete Institute. The cement had a fineness of about 89 per cent passing the No. 200 sieve and

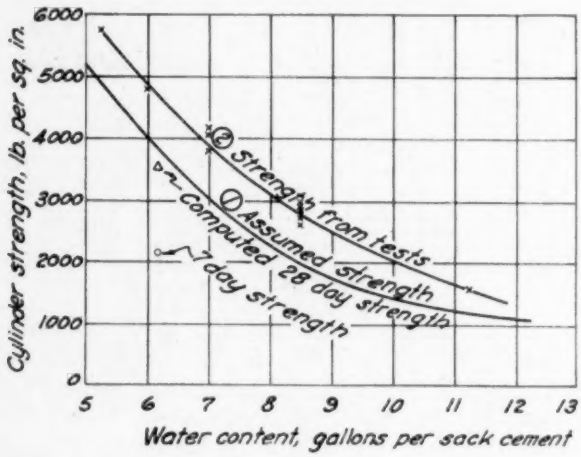

FIG. 4—WATER-CEMENT-STRENGTH RATIO, (1) ASSUMED FOR DESIGN AND (2) SECURED FROM TESTS

gave a high 28-day strength. The aggregates came from near the Delaware River at Morrisville, Pennsylvania. The sand had an average fineness modulus of about 2.82, had 4 per cent passing the No. 48 sieve, and about 50 per cent between the No. 48 and No. 28 sieves. The sand content used was 40 per cent by weight of the total aggregates. This combination gave very workable concrete for all mixes included in the series. The coarse aggregate consisted of gravel of a maximum size of ¾ in. At the laboratory it was screened and recombined so as to contain 40 per cent between the No. 4 and the ⅜-in. sieves, and 60 per cent between the ⅜-in. and the ¾-in. sieves. The aggregates had been stored indoors some time before the making of the concrete. At the time of making specimens, the gravel contained about 0.5 per

TABLE 2—CONCRETE MIXES USED IN BEAMS

Group No.	Concrete Mix By		Water-Cement Ratio By		Cement Content Sacks Per Cu. Yd.	Water Content Gallons Per		Slump of Conc. In.
	Weight	Abs. Vol.	Weight	Abs. Vol.		Sack Cement	Cu. Yd. Conc.	
1	1–3.80–5.70	1–4.55–6.80	1.00	3.10	3.6	11¼	40.5	5
2	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	5½
3	1–2.30–3.45	1–2.75–4.15	.62	1.92	5.7	7	39.9	5½
4	1–1.90–2.85	1–2.25–3.40	.53	1.65	6.7	6	40.2	7½
5	1–1.60–2.40	1–1.90–2.85	.46	1.44	7.7	5¼	40.4	6½
6	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	5
6A	1–2.30–3.45	1–2.75–4.15	.62	1.92	5.7	7	39.9	3
7	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	4
8	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	6
9	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	5
10	1–2.80–4.20	1–3.35–5.00	.75	2.33	4.8	8½	40.8	5
10A	1–2.30–3.45	1–2.75–4.15	.62	1.92	5.7	7	39.9	5

cent water as moisture and the sand about 1 per cent. It was assumed that this was close to the percentage which would be absorbed and no correction for absorption was made.

At the time of designing the mix only the 7-day strength of concrete of one water content was available. This strength was converted into estimated 28-day strength by use of the formula $f_{28} = f_7 + 30\sqrt{f_7}$, in which f_7 and f_{28} are the 7-day and the 28-day strengths. An estimated water-cement-strength curve was drawn so that it passed a little above this estimated strength (see curve 1, Fig. 4). Fig. 4 also shows the strengths obtained from the tests and indicates that for the cement used the strength increase between the ages of 7 and 28 days was considerably greater than the increase given by the formula. After the water content per sack of cement for each strength of concrete had been taken from curve 1, a few trial batches were made with one of these water contents to determine the mix which would give suitable workability. The mix found to give the proper workability contained 40 gal. of water per cubic yard of concrete and this mix was used as the basis for designing the other mixes. It was assumed that the water content per cubic yard of concrete remained practically constant for mixes of a given consistency. All mixes were therefore designed so as to contain 40 gal. of water per cubic yard for the different water-cement ratios used. The mixes arrived at in this manner are given in Table 2 which contains all data of the concrete used in the series. In Fig. 5 the slumps have been plotted as ordinates of two independent graphs.

In one of them the water content, shown at the top, and in the other the cement content, shown at the bottom, represent the abscissas. For the resulting concrete 75 per cent of the slumps lay between 4 and 6 in. The extreme range was from 3 to 7½ in. As large a variation as this may be expected on account of lack of uniformity in the moisture content of the aggregates.

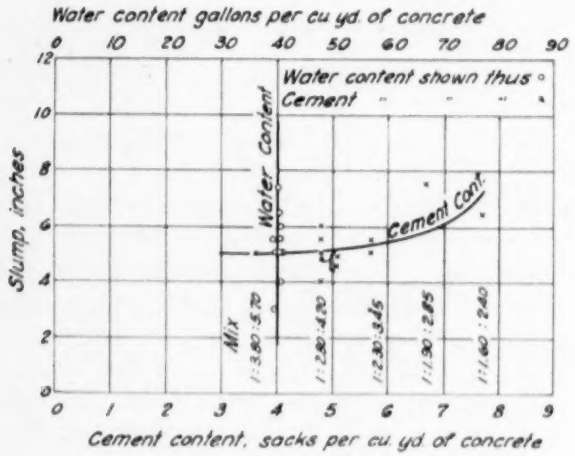

FIG. 5—RELATION BETWEEN WATER CONTENT, CEMENT CONTENT, AND SLUMP OF CONCRETE

If the assumptions made were entirely correct and if the operation had been under perfect control, the curve representing the relation between slump and cement content should be a straight horizontal line. Due to variation in slump already pointed out, this condition was only partially realized as is shown in Fig. 5. However, the conformity was good enough to warrant the characterization of this method as a satisfactory working basis.

6. *Cement-Strength Relation.* In Fig. 6 28-day cylinder strengths of the concrete have been plotted as ordinates of two independent curves with the water content and cement content per cubic yard of concrete as abscissas. In this series, however, the water content was held constant and the relation between strength and water content was necessarily a vertical straight

line. It will be seen from this figure that the cement strength relation is approximately a straight line, that is, the increase in strength was proportional to the increase in cement content. It should be kept in mind that the same aggregate was used throughout the series and that the proportion of sand to coarse aggregate was constant, 40 per cent fine and 60 per cent coarse, and that

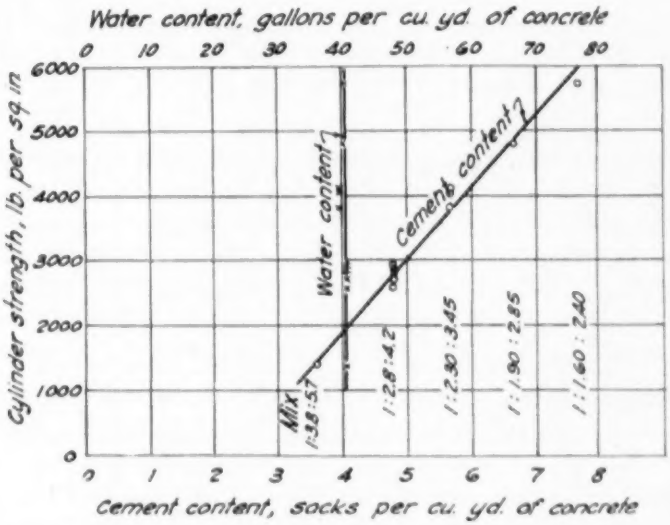

FIG. 6—RELATION OF STRENGTH TO CEMENT CONTENT AND WATER CONTENT FOR CONCRETE OF CONSTANT CONSISTENCY

the consistency of the concrete was fairly constant as measured by the slump. The increase in strength was approximately 1100 lb. per sq. in. per sack of cement per cu. yd. of concrete.

It should not be expected that with variations in types of aggregate or with variation of proportion of fine to coarse aggregate in different mixes, a constant water content should give a constant consistency or that proportionality between increase in cement content and increase in strength should result.

7. *Uniformity of Concrete.* Even though each cylinder represented a different batch of concrete in a beam, the variation in strength between individual cylinders for a given beam was less than the variation between the averages for beams of the

same group. Therefore, in studying the uniformity of concrete the average strength of the three cylinders from each beam was considered as a single strength. The maximum variation of these strengths from the average of a group was computed for each group of three beams. The largest of these maximum variations for any group in the entire series was 12.9 per cent and the average of the maximum variations for all groups was 6.4 per cent. Table 3 gives the average cylinder strength for each beam and also contains the average strength for each group and the maximum variations from the average.

8. *Reinforcement.* The reinforcement bars were of rail steel which gave high yield-point stress and high ultimate strength. Table 4 shows the test results for coupons from the reinforcement bars. The average yield-point stress for all specimens was found to be 64,800 lb. per sq. in., the ultimate strength 106,400 lb. per sq. in., and the modulus of elasticity 29,500,000 lb. per sq. in. This modulus of elasticity was used in computing the ratio n, that is, the ratio of the modulus of elasticity of steel to that of concrete.

MAKING, CURING AND TESTING OF SPECIMENS

9. *Fabrication of Reinforcement.* The longitudinal reinforcement was in one layer in some and in two layers in others of the beams. The clear distance between the two layers of bars was ¾ in. In order to hold the reinforcement in its proper position with respect to the bottom of the beam, supporting legs of the necessary depth were welded to the lower layer of the longitudinal bars.

The proper horizontal spacing was maintained by means of short cross bars welded to each longitudinal bar of the layer.

The web reinforcement was looped around and attached to the longitudinal reinforcement by electric arc welding. Generally, where two layers of longitudinal bars were used the stirrups were looped around the bars of the lower layer and welded to both layers of the longitudinal reinforcement, thus insuring the proper spacing and position of stirrups and longitudinal bars.

For simplification in making up stirrups the same depth of stirrups was used for beams of various depths, as far as possible. During the making up of the beams it appeared that the stirrups

were too short to be adequate as web-reinforcement for beams with an effective depth of 14 inches. To correct for this deficiency in some measure these stirrups were looped around and welded to the bars of the upper layers in two out of three of the 14-in. beams instead of to bars of the lower layer. The upper and lower layers of bars in these two beams were tied together by means of spacers welded to both bars. For beams whose depth was too small to provide adequate anchorage for vertical stirrups inclined web bars were used. These also were welded to the longitudinal reinforcement, which in these cases included only one layer of bars.

This method of fabricating all the reinforcement for each beam into a rigid unit appeared to be very effective in maintaining the correct position of the reinforcement during the placing of concrete in the beam. It also reduced the danger of slipping of the reinforcement during the testing of the beam. Fig. 7 shows the fabricated reinforcement for beams of different depths.

10. *Forms.* For the beams having an effective depth of 8 in. or less, wooden forms were used. For those having a depth of 10 in. or more, steel forms were used. In order to prevent leakage from the forms, all joints were made with care, and as a further precaution, all joints (with both wooden and steel forms) were puttied before placing the concrete. In this way the amount of leakage was made very small. Before placing the concrete all forms were oiled as a precaution against sticking of the concrete to the forms. In the case of the wooden forms oiling was a precaution also against absorption of water and warping of the forms.

11. *Mixing and Placing.* The concrete was mixed in an inclined kettle-shaped mixer of approximately 2.5 cu. ft. capacity. The dry materials were dumped into the mixer and mixed for one minute; water was then added and the mixing was continued for three minutes more. The concrete was dumped into the metal container shown in Fig. 8 and shoveled from it into the forms. The rounded ends of this container facilitated the complete emptying of it. Each beam required more than one batch. The concrete of each batch was placed in the form while the succeeding

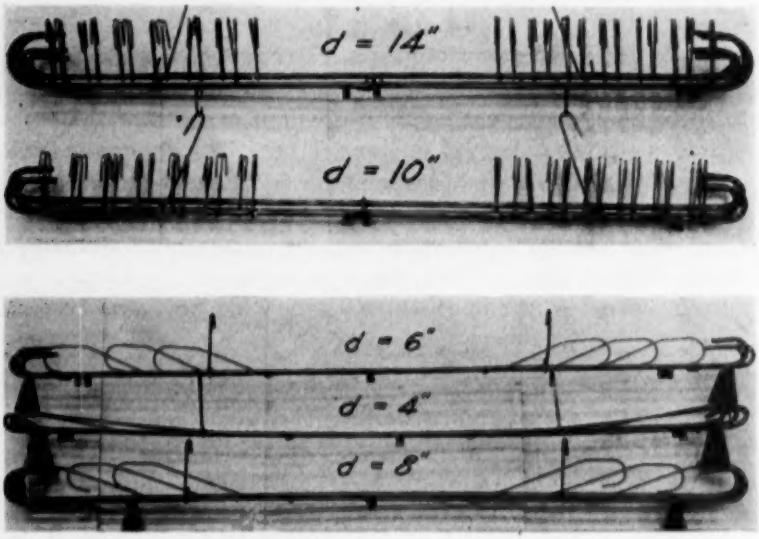

FIG. 7—REINFORCEMENT FABRICATED FOR BEAMS

batch was being mixed. Each batch was spread to an approximately uniform depth over the full length of the beam. The concrete was rodded into place by means of a ⅝-in. steel bar, and the forms were hammered lightly in order to consolidate the mass. Generally one control cylinder was taken from each batch so as to represent the different layers of the concrete in the beam. Fig. 8 shows the arrangement during the making of the beams in which wooden forms were used.

12. *Curing.* The control cylinders, as well as the beams, were generally left in the forms for 48 hr. after making. The forms were then removed and the specimens placed in a moist room of 100 per cent humidity and a temperature of approximately 70°F, until a 28-day age was reached. A typical chart of the temperature in the moist room covering one week of the curing, February 24 to March 3, is shown in Fig. 9. The temperature range throughout the curing period was from 68° to 72°F.

13. *Testing.* At the age of 28 days the specimens were removed from the moist room and tested to failure. The testing machine used, both for the control cylinders and the beams, was an Olsen screw-power machine equipped with supporting wings

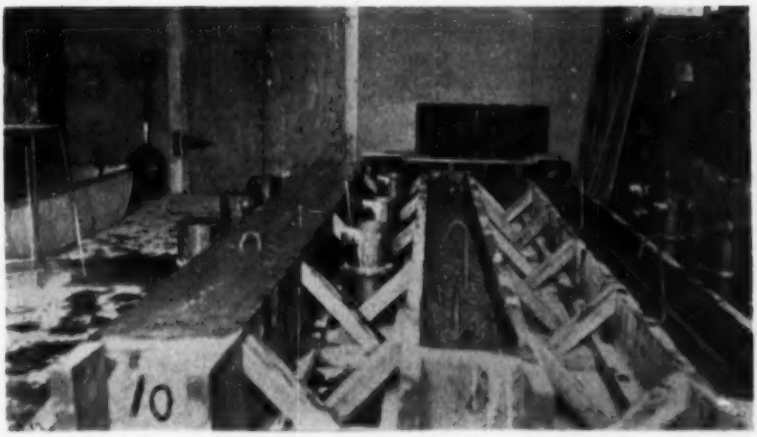

FIG. 8—MAKING THE BEAMS

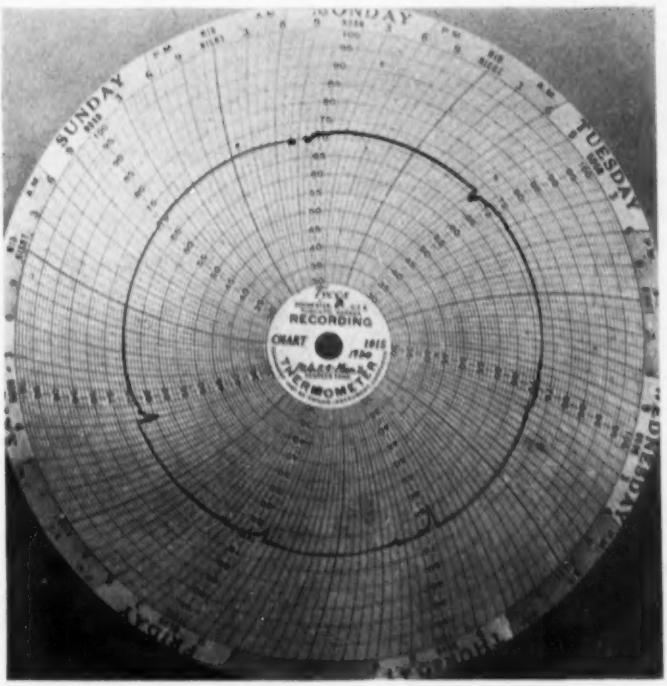

FIG. 9—RECORD OF TEMPERATURE IN MOIST ROOM FOR WEEK FEB. 24—MARCH 3, 1930

designed for the testing of beams. The 6 by 12-in. cylinders were tested in the ordinary manner, with an idling speed of the head of the machine of .05 in. per minute. The beams rested on a roller at one support and on a spherical bearing block at the other support. The center of each support was 9 in. from the end of the beam, making a distance of 9 ft. 6 in. between centers of supports. The load was transferred from the head of the machine to the concrete beam through a heavy structural steel loading beam having a reinforced web. This loading beam transferred the load through a roller at one end and a spherical bearing block at the other end to the test beam at the load points, which were placed 21 in. on either side of the center of the beam. This arrangement of loads produced a constant moment in the central 42-in. portion of the concrete beam. The spherical bearing blocks at one support of the test beam and at one load-point were introduced for the purpose of eliminating torsional moments and eccentric loading in the testing of the beams. Fig. 10 shows the arrangement during the testing of one of the beams.

During the testing of the beams close examination was made for the appearance of cracks.

14. *Deformation Measurements.** Deformation measurements were taken on at least one control cylinder from each beam. Two metal collars surrounding the cylinders at sections 10 in. apart were attached to the cylinder by means of a pair of screws in each collar 180 degrees apart. All four screws lay in the same diametral plane. A steel rod used as a distance piece held the collars a fixed distance apart at a point on each collar 90 degrees from the axis of the screws. This arrangement allowed each collar to rotate about a horizontal axis through the screws as the cylinder shortened under the load. Opposite this distance piece a .0001-in. Ames gage was attached to the upper collar, and another distance piece, attached to the lower collar bore at its upper end upon the plunger of the Ames gage. The readings on the dial, therefore, indicated twice the amount of the shortening of the cylinder during the testing. The deformation results secured in this manner were used for the determination of the modulus of elasticity of the concrete.

*In this report: (1) the word "deformation" denotes total change of length in a given gage length. (2) The word "strain" denotes the change per unit of length which is caused by stress. (3) The word "stress" denotes the intensity per unit area of the internal force.

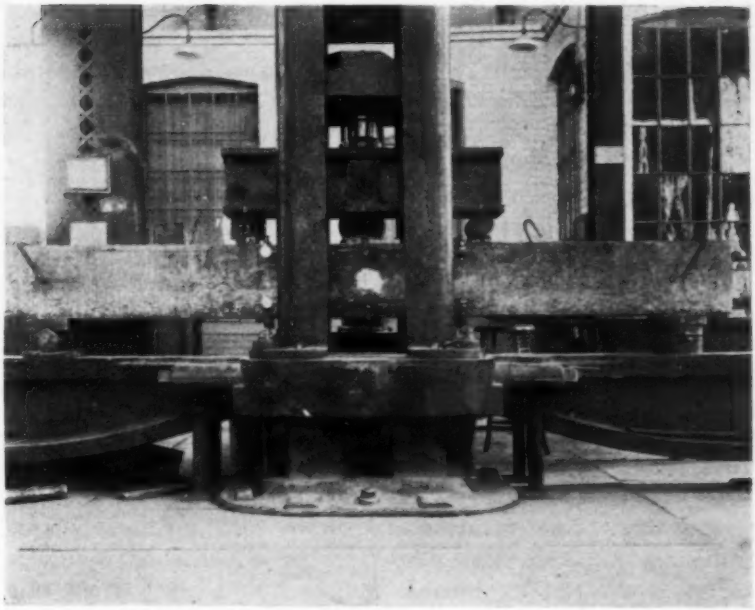

FIG. 10—TESTING ARRANGEMENT

Table 3 gives the modulus of elasticity for the concrete in each group of beams. This modulus is the tangent modulus at a stress of 500 lb. per sq. in. This table also gives the corresponding values of n for each group.

The deformations of the concrete in the beams were measured on each side of the beam by means of Ames gages reading generally 0.001 in. One gage line was placed near the top of the beam, one near the elevation of the center of gravity of the reinforcement, and in some cases, one was placed near the neutral axis of the beam. The gage length was 35 in. for the gages near the top and near the center of gravity of the reinforcement, and 30 in. for the gages near the neutral axis. The Ames gages used for the observation near the neutral axis read to 0.0001 in. The deformation apparatus may be seen in Fig. 10.

15. *Observed Deflections.* For measuring the center deflection a fine wire attached to the side of the beam near the neutral axis

TABLE 3—PROPERTIES OF CONCRETE IN BEAMS

Group No.	Depth *d*, In.	Ratio of Reinforcement, *p*	Cylinder Strength, Lb. Per Sq. In. for Beam				Max. Var. From Av. Per Cent	Modulus of Elasticity Lb. Per Sq. In.	Values of *n*
			A	B	C	Av.			
1	10.2	.021	1210*	1520	1450	1390	12.9	2,650 000	11.1
2	10.3	.028	2530*	2940	2910	2790	9.3	3,450 000	8.6
3	10.3	.037	4020	4200	4000	4070	3.2	3,750 000	7.9
4	10.1	.047	4670	4660	5060	4800	5.4	4,150 000	7.1
5	10.2	.056	5680	5680	5870	5740	2.3	4,620 000	6.4
6	14.2	.030	2490	2600	2670	2590	3.9	3,650 000	8.1
6A	14.1	.039	3990	4120	4270	4130	3.4	3,800 000	7.8
7	12.2	.028	2800	2860	3200	2950	8.5	4,400 000	6.7
8	8.0	.031	3020	2650	2800	2760	9.4	3,650 000	8.1
9	5.9	.032	3120	2670	2900	2900	8.0	3,200 000	9.2
10	4.1	.030	3040	2750	2660	2820	7.8	3,475 000	8.5
10A	4.1	.040	3730	3900	3800	3810	2.4	3,725 000	7.8
						Average	6.4		

*Extra water added for absorption.

at each support spanned the length of the beam. A mirror bearing a scale graduated to fiftieths of an inch was attached to the side of the beam at the center of the span at the elevation of the neutral axis. The elevation of the wire on the scale was read without parallax by lining up the wire with its image in the mirror at the time of reading the scale. The deflections thus observed were read by estimation to the nearest .01 in. Fig. 10 shows the arrangement for the deflection measurements during the testing.

16. *Duration of Test.* The time required for the testing of one beam, that is, the time between initial load and the maximum load, depended upon the strength of the beam and the number of deformation and deflection measurements made. In general, the testing was completed in less than one-half hour. Beams carrying a maximum load of more than 20,000 lb. were loaded in 5000-lb. increments between successive sets of readings, while for the beams of lower carrying capacity smaller increments between successive sets of observations were used.

RESULTS

17. *Occurrence of Cracks.* During the testing of the beams close inspection was made for the appearance of cracks. Usually the first cracks appeared at the bottom of the beam just below the loading point or near the center of the span. Usually no cracks appeared in the portion of the beam between the supporting points and the loading points. However, a few beams

showed diagonal tension cracks between the supports and the loading points. These were the beams of groups 6, 6A and 7, having an effective depth of 12 and 14 in. which showed diagonal tension cracks prior to failure. These cracks occurred between the supports and the loading points at both ends of the beam. Generally, several cracks occurred parallel to each other and only a few inches apart. In these cases the cracks ordinarily extended from the neutral axis to the bottom of the beam. Most of these cracks were first discovered near the bottom, but some appeared first at approximately the level of the neutral axis. The load at which the first crack was observed is given in Table 5 which also gives the percentages of the maximum load at which the first crack was observed. The average load at which the first crack was observed was about 48 per cent of the maximum load for all the beams.

18. *Forms of Failure.* All beams except two failed in compression of the concrete between the loading points of the beam. Generally the failure was very close to the center. Beams of low strength concrete gave a gradual failure, while beams of higher strength concrete broke suddenly without previous warning.

The other two beams failed in diagonal tension. These were from the series in which the average strength of the concrete was 2800 lb. per sq. in., and in which the effective depth varied from 4 to 14 in. The effective depth was 14 in. for one of these beams, and 8 in. for the other. The 14-in. beam which failed in this manner was the one which had stirrups of 11-in. depth attached to the lower layer of the longitudinal steel reinforcement. The stirrups in this beam extended only partially into the compression area of the concrete. In the other two beams of the same group the stirrups were attached to the longitudinal bars of the upper layer. These beams failed in compression, even though diagonal tension cracks appeared before failure. The maximum loads, shown in Table 5, indicate that the compressive strength of the concrete was almost reached before the diagonal tension failure occurred. Fig. 11 is a photograph of the 14-in. beam which failed in diagonal tension.

Beam 8B, 8 in. deep, had inclined bars and failed in diagonal tension without previous warning. The two companion beams

TABLE 4—SUMMARY OF TESTS ON REINFORCING BARS USED IN BEAMS

Folio No.	Nom. Bar. Size In.	No. of Tests Made	Meas. Area* Sq. In.	Yield Point** Lb. Per Sq. In.	Ultimate Strength Lb. Per Sq. In.	Elong. in 8" Per Cent	Reduc. in Area† Per Cent	Modulus of Elasticity‡ Lb. Per Sq. In.
7555	⅜	4	.110	72,250	108,400	13	38	29,700,000
7555	½	2	.427	73,100	117,700	9	20	28,400,000
7555	⅞	4	.593	67,400	108,600	12	26	29,400,000
7555	1	4	.751	62,650	111,400	14	30	28,850,000
	Average			68,240	110,500	12.4	30	29,180,000
7576	⅜	4	.101	74,400	113,200	12	44	30,200,000
7576	½	4	.422	57,950	95,800	16	30	29,500,000
7576	⅞	4	.591	58,400	101,800	19	36	30,000,000
7576	1	4	.756	55,850	100,500	15	24	29,300,000
	Average			61,400	102,800	15.5	33.5	29,750,000
	⅜	8	.105	73,400	110,800	12.5	41	29,950,000
	½	6	.424	63,000	103,100	14	27	29,130,000
	⅞	8	.592	62,900	105,200	15.5	31	29,700,000
	1	8	.753	59,300	106,000	14.5	27	29,300,000
Grand Average		30		64,800	106,400	14	32	29,480,000

Notes: *Computed from the weight and length of bar.
**Yield point detected by the drop of the beam in only half of the tests made.
†Reduced area computed from an average reduced diameter.
‡Strains were measured in only half of tests made.

TABLE 5—RESULTS OF BEAM TESTS

Group No.	Dimensions of Beams		Actual Ratio of Reinforcement, p	Maximum Load on Beams, Lb. Beam No.				Max. Var. From Ave. Per Cent	Load at First Crack Per Cent**	Max. Shearing Stress Lb. Per Sq. In.†
	Depth d in.	Width b in.		A	B	C	Av.			
1	10.2	8.2	.021	33000	32000	32600	32520	1.6	52	243
2	10.3	8.2	.028	47390	40000	46750	44710	10.5	36	321
3	10.3	8.2	.037	58550	64770	66580	63300	7.5	43	452
4	10.1	8.2	.047	74500	71050	78950	74830	5.5	37	546
5	10.2	8.3	.056	84350	96410	81470	87410	10.3	38	625
6	14.2	8.2	.030	92000*	106570	92000	99290	7.3	63	522
6A	14.1	8.2	.039	120000	113600	130000	121200	7.3	37	634
7	12.2	8.3	.028	69250	63870	71340	68150	6.3	47	407
8	8.0	8.1	.031	25760	31650*	33430	29600	13.0	54	280
9	5.9	7.9	.032	15270	18640	16580	16830	10.8	65	220
10	4.1	8.0	.030	6790	7670	6220	6890	11.3	58	128
10A	4.1	8.0	.040	9750	10300	9000	9680	7.0	41	180
							Av.	8.2	48	

*Failed in diagonal tension, omitted from average.
**Percentage of maximum load.
$\dagger V = \frac{W}{2jbd}$

of the same group, 8A and 8C, failed in compression with no diagonal cracks or other indication of being near failure in diagonal tension. It is to be noted also, that the maximum load was slightly higher for beam 8B, which failed in diagonal tension, than the average load for 8A and 8C, which failed in compression. In order to investigate whether any slipping of the inclined bars took place, thus causing the diagonal tension failure, the concrete in beam 8B was removed so that the inclined bar could be in-

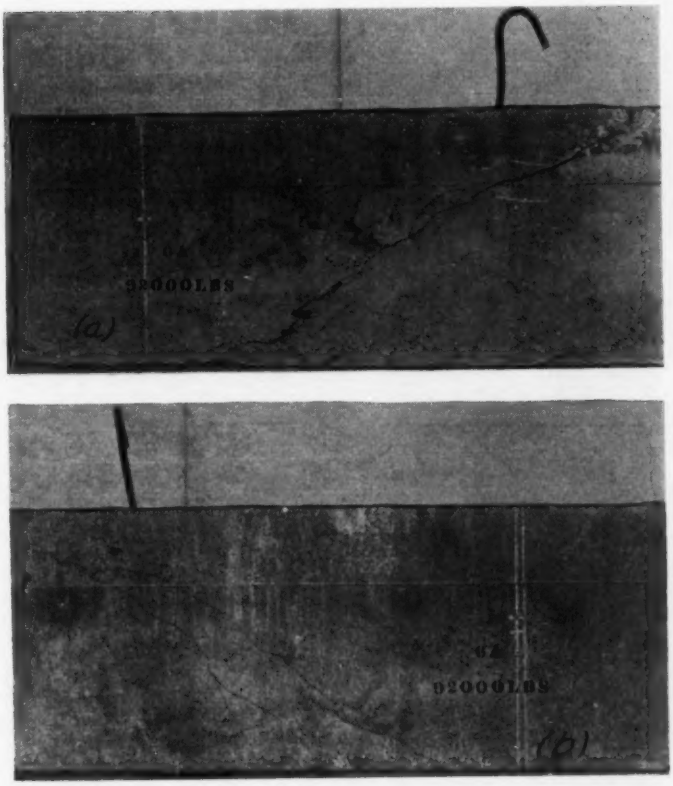

FIG. 11—DIAGONAL FAILURE OF BEAM 6A. (A) LEFT END, (B) RIGHT END

spected throughout its length. No indication of slipping of the bar or stressing the steel beyond the yield point could be detected. The welded connection of the inclined to the longitudinal bar was still intact and there is no evidence that the diagonal tension failure was due to slipping of the web reinforcement or to failure of the welded connection with the longitudinal bar. Fig. 12a shows beam 8B in the testing machine after it had failed and 12b, photographed from the opposite side, shows the inclined bar exposed throughout its length.

Typical examples of the forms of failure for the different beams are shown in Fig. 13, 14 and 15. Fig. 13 shows the beams of

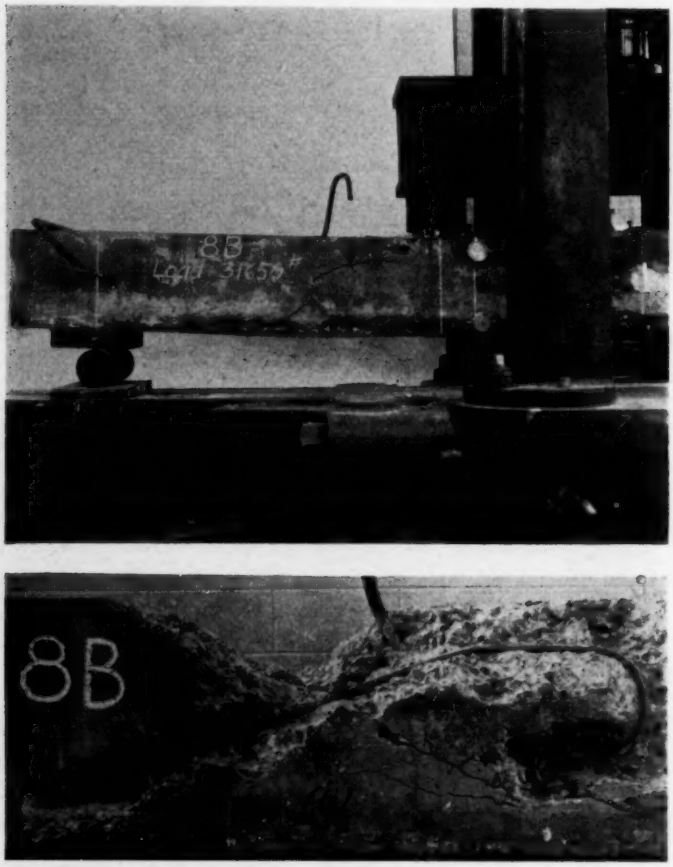

FIG. 12—DIAGONAL FAILURE OF BEAM 8B. (A) IN TESTING MACHINE, (B) WITH EXPOSED INCLINED BAR

equal dimensions but of different strengths of the concrete; Fig. 14 the beams of 2800 lb. per sq. in. concrete and different depths of beams and Fig. 15 beams of 4000 lb. per sq. in. concrete and different depths.

19. *Prism Tests.* In order to get more definite information regarding the distribution of strains and stresses in the compression section of the concrete area in the beam, two 8 by 8 by 12-in. concrete prisms were made during the manufacturing of

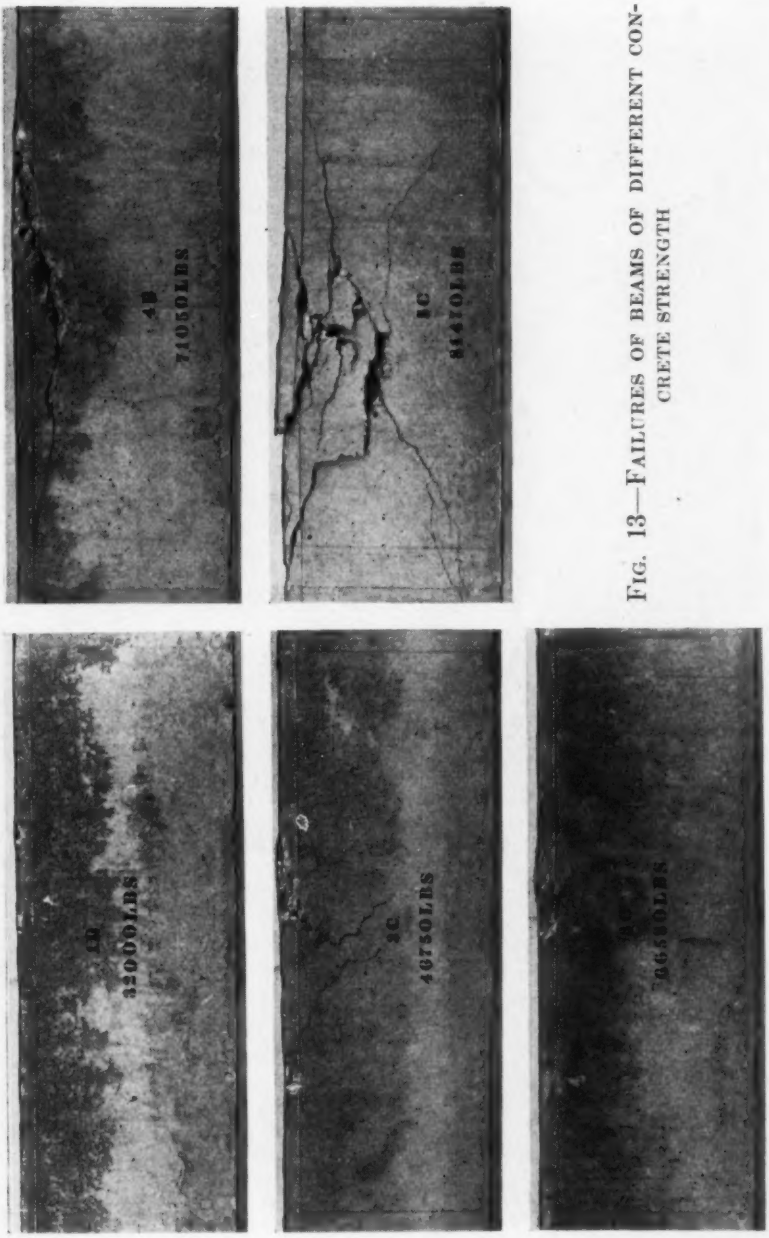

FIG. 13—FAILURES OF BEAMS OF DIFFERENT CONCRETE STRENGTH

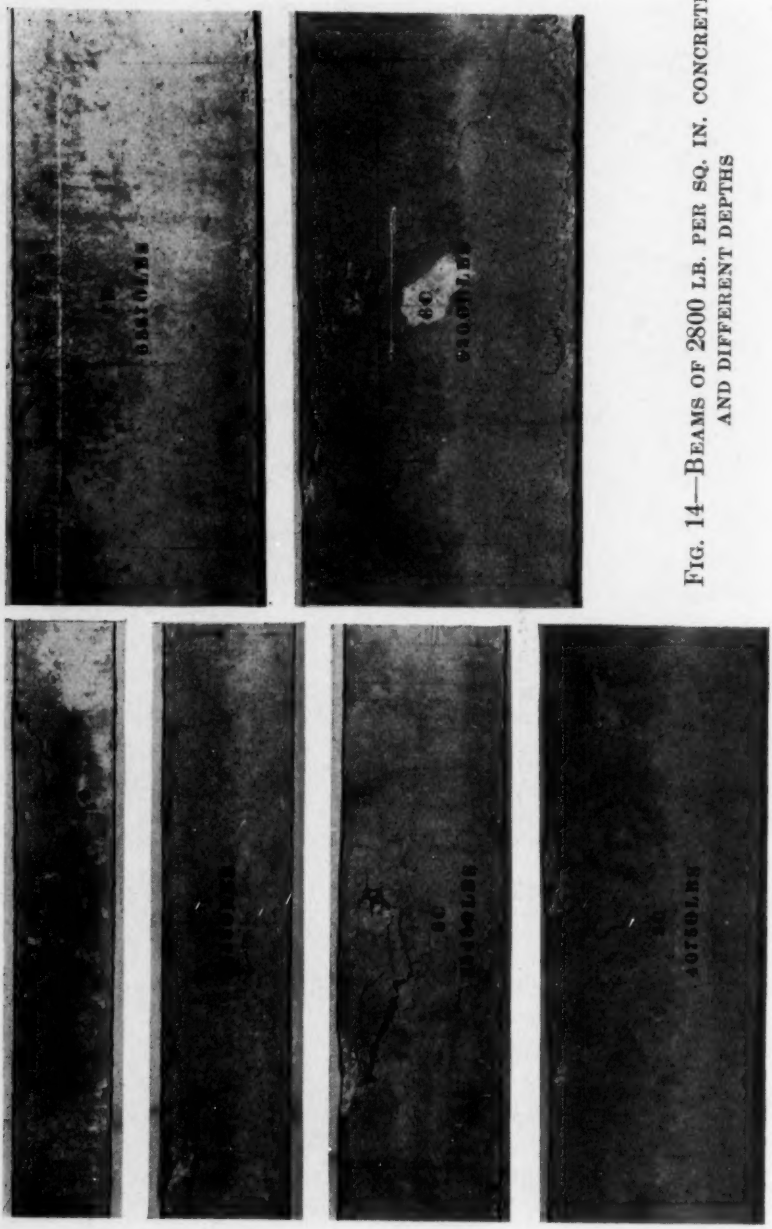

FIG. 14—BEAMS OF 2800 LB. PER SQ. IN. CONCRETE AND DIFFERENT DEPTHS

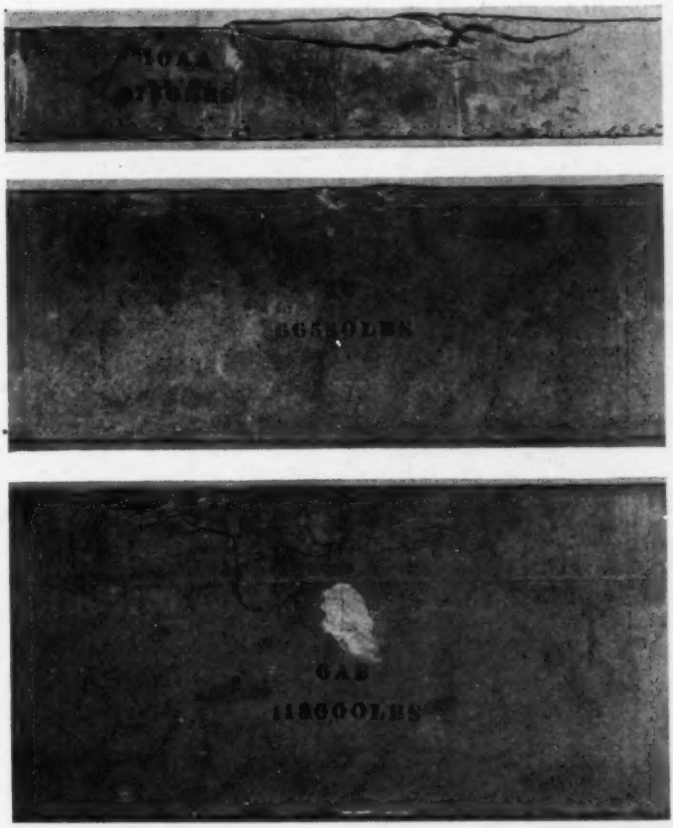

FIG. 15—BEAMS OF 4000 LB. PER SQ. IN. CONCRETE AND DIFFERENT DEPTHS

the beams. After all the beams had been made three prisms and three cylinders were made from a mix of concrete similar to that used in group 2 of the beam series. All the prisms were tested at an age of 28 days in such a way that the resultant load and the resultant reaction passed through the edge of the middle one-third of the sectional area of the prism. Fig. 16 which shows one of the prisms after failure indicates the method of applying the loads.

Deformation measurements were taken on two opposite faces of each prism.

The average strains for the two prisms from beams A and B in group 6A are plotted in Fig. 17 by connecting by straight lines the points representing the strains measured three inches away from the opposite faces, that is, assuming a straight line variation of strains across the section of the prism. The deformation curves for low loads all seem to pass through the same point, and show that there was little or no strain at one edge of the prism. This indicates that for these loads the relation between strain and stress is represented very closely by a straight line, since it is only for a straight line stress-strain relation that the strain is zero at the edge of a prism loaded at the edge of the middle third of the sectional area. For larger loads, however, the point of zero strain lay slightly within the prism, indicating that a curved stress-strain distribution existed at high stresses. Fig. 18 shows the average stress-strain diagram from the same data as in Fig. 17 for that edge of the prism which lay nearest to the load-point. It also shows the average stress-strain diagram for the control cylinders from the same batches of concrete. This figure indicates that the modulus of elasticity

FIG. 16—8 BY 8 BY 12-IN. PRISM LOADED AT EDGE OF MIDDLE THIRD OF CROSS SECTION

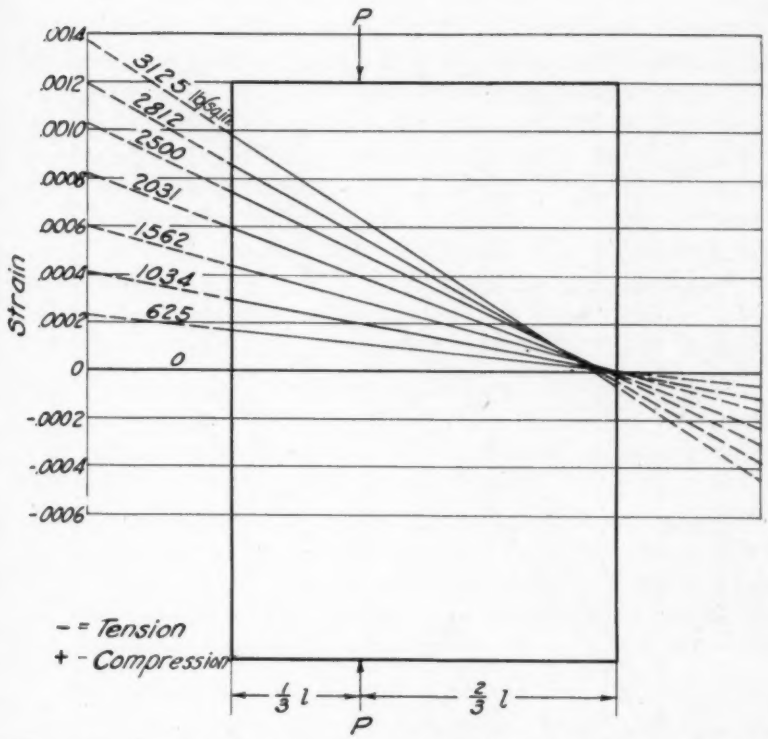

FIG. 17—STRAIN DISTRIBUTION IN CONCRETE PRISM LOADED AT THE EDGE OF THE MIDDLE THIRD

obtained for the prism was slightly less than that found on the control cylinders. The average cylinder strength for these two beams represented in Fig. 17 and 18 was 4060 lb. per sq. in.

The deformations found in the additional prisms made after completion of the making of the beams were quite similar to those already discussed.

20. *Uniformity of Results.* The uniformity in strength results obtained from the tests of three beams in each group was found to be very good. The maximum variations of the individual strengths from the average are shown in Table 5. The greatest of these maximum variations was 13 per cent, and the average for all the groups was 8.2 per cent. The average of the maximum

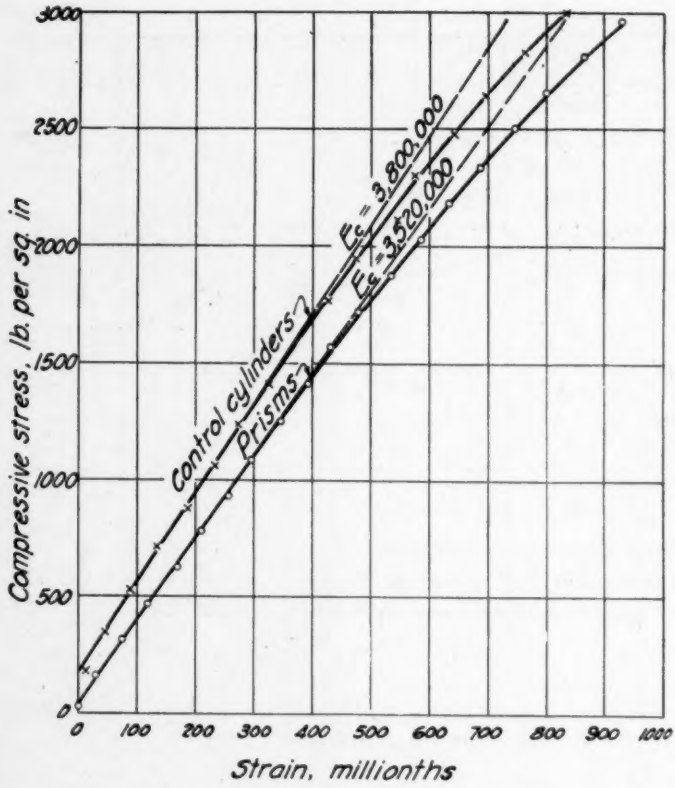

FIG. 18—STRESS-STRAIN DIAGRAM FOR PRISMS OF GROUP 6A, AND CONTROL CYLINDERS

variations for the beams was slightly higher than the average found for the concrete cylinders. The relatively small variation between the individual specimens indicates that the beams were uniform in quality and that the results represent the actual strength fairly well. A further indication of the reliability of the results is found in the fact that the points showing the beam-cylinder strength ratios for beams of different concrete strengths and for beams of different heights, fall approximately on smooth curves.

21. *Relation between Strength of Cylinders and Strength of Prisms.* The two prisms made from the same concrete as the beams in group 6A, loaded eccentrically, gave an average compressive strength over the entire area, of 2430 lb. per sq. in. Assuming a straight line stress-strain relation this corresponds to a maximum stress at one face of the prism of 4860 lb. per sq. in., and zero stress at the opposite face. The control cylinder made from the same batches and tested with a uniformly distributed load, gave an average compressive strength of 4060 lb. per sq. in. The apparent strength of the concrete in the prisms was therefore about 20 per cent higher than that of the cylinders.

The three prisms which were made at a later date did not have perfect caps, and therefore failed at lower stresses than might otherwise have been the case. The average compressive strength over the entire area of these prisms was 1675 lb. per sq. in. This corresponds to a maximum fiber stress of 3350 lb. per sq. in. The average strength of the control cylinders from the same batch was 3110 lb. per sq. in. The maximum fiber stress in these prisms was therefore only 8 per cent greater than the cylinder strength of the concrete.

22. *Relation between Beam Strength and Prism Strength.* This comparison includes only the two prisms made from the same batches of concrete as the beams A and B in group 6A. If a straight-line stress-strain distribution is assumed for the prism, it is found that the prism strength was about 20 per cent higher than the cylinder strength. Using the straight-line formula for computing stresses in the beams, and using the value of n obtained from the deformation measurements on the control specimens of concrete and of steel, the computed ultimate stress in the beam was about 50 per cent greater than the strength of the cylinders, and 25 per cent greater than the ultimate stress in the prism. The ultimate fiber stress in the prism was therefore considerably less than the fiber stress computed for the beams.

23. *Notation and Formulas.* For the purpose of studying the test results, an analysis has been made in which the stress curve in the compression part of the beam is assumed to be a parabola, not necessarily of second degree, but of some degree, r. The value

of r is always the ratio of $E_c \cdot \epsilon'_c$ to f''_c. See Fig. 19. When r is one, the stress curve reduces to a straight line and the resulting formulas to the ordinary straight line formulas. When r is two, the stress curve reduces to the parabola of second degree and the formulas to the ordinary parabolic formulas for stress at the load

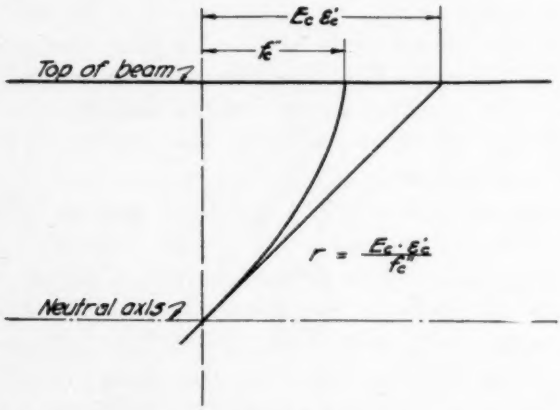

FIG. 19—ASSUMED STRESS DISTRIBUTION IN COMPRESSION AREA OF BEAM

which produces compression failure in the concrete. The analysis could be made applicable for loads below the maximum, but there seems to be no necessity for introducing this complication.

In the formulas used in this report the following notation will be used.

b = width of beam

d = effective depth of beam, that is, depth from compression surface of beam to center of gravity of sectional area of reinforcing steel

p = ratio of reinforcement

n = ratio of modulus of elasticity of steel to that of concrete

M = bending moment at maximum load

kd = vertical distance from compression surface to neutral axis

jd = distance from resultant compressive force to resultant tensile force, that is, moment arm of resisting couple

f_c'' = computed compressive stress in extreme fiber of reinforced concrete beam at load causing compressive failure

f'_c = compressive strength of concrete as determined from tests on 6 by 12-in. cylinders

f_s = computed stress in reinforcement of concrete beam

r = exponent in a stress-strain equation (assumed to be parabolic) whose graph approximates the stress-strain curve for the control cylinders

The general formulas for k, j, f''_c, and f_s are as follows:

$$k = \sqrt{pn(r+1) + \left(\frac{pn(r+1)}{2}\right)^2} - \frac{pn(r+1)}{2}$$

$$j = 1 - \frac{r+1}{2(r+2)}\,k$$

$$f_c'' = \frac{r+1}{r}\,\frac{M}{kjbd^2} \text{ and}$$

$$f_s = \frac{M}{pjbd^2}$$

The condition for straight-line distribution of stress is obtained when r = 1. The equation for stress in the concrete at the compression surface then becomes $f''_c = \frac{2M}{kjbd^2}$. The condition for parabolic distribution of stress is obtained when $r = 2$. The equation of stress, f''_c, in the concrete at the compression surface then becomes $f''_c = \frac{3}{2}\,\frac{M}{kjbd^2}$

An illustration of the effect of the variation in the assumptions as to the stress distribution on the intensity of the maximum stresses and the position of the neutral axis is given in Fig. 20. In this figure the curves show the stress distribution for straight line, second degree parabola, and fourth degree parabola respectively.

24. *Effect of Variation in Strength of Concrete on Beam-Cylinder Strength Ratio.* Table 6 shows the extreme fiber stresses in the beams at maximum load as computed by the straight-line formula. In these computations three different values of n have been used, (1) the values specified by the American Concrete

TABLE 6—BEAM-CYLINDER STRENGTH RATIO USING STRAIGHT LINE STRESS DISTRIBUTION, $r = 1$

Group No.	Depth d, In.	Average Cylinder Strength Lb. Per Sq. in.	Stress Computed By Straight Line Formula; Value of n According to			Beam-Cylinder Strength Ratio for n According to		
			A. C. I.	J. C.	Observations	A. C. I.	J. C.	Observations
1	10.2	1390	2870	3110	3350	2.06	2.24	2.41
2	10.3	2790	4220	4120	4490	1.51	1.48	1.61
3	10.3	4070	6120	5700	6030	1.50	1.40	1.48
4	10.1	4800	7410	6650	7160	1.54	1.39	1.49
5	10.2	5740	8410	7280	8000	1.46	1.27	1.39
6	14.2	2590	4780	4730	5240	1.85	1.83	2.02
6A	14.1	4130	6220	5770	6110	1.50	1.40	1.48
7	12.2	2950	4600	4630	5150	1.56	1.57	1.75
8	8.0	2760	4570	4470	4950	1.66	1.62	1.79
9	5.9	2900	4900	4740	5060	1.69	1.63	1.75
10	4.1	2820	4170	4060	4310	1.48	1.44	1.53
10A	4.1	3810	5900	5570	5870	1.55	1.46	1.54

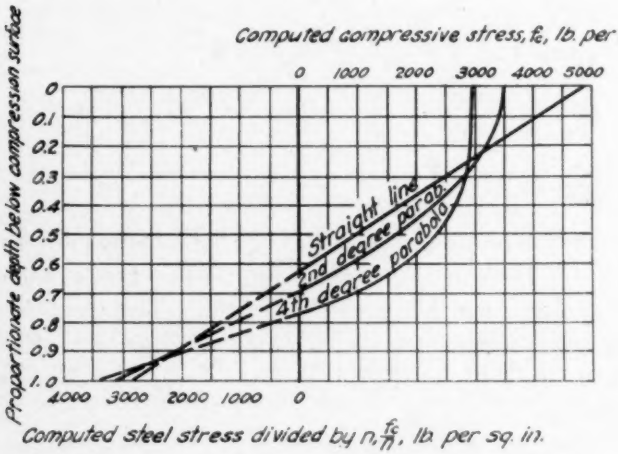

FIG. 20—STRESSES IN BEAMS ACCORDING TO VARIOUS ASSUMPTIONS AS TO DISTRIBUTION OF COMPRESSIVE STRESS; P = .057, N = 10

Institute Code of 1928,* (2) the values specified in the Joint Committee report of 1924,† (3) the values determined from the deformation measurements on control cylinders and on coupons from the reinforcement steel used in the tests. The compressive strengths of the concrete as obtained from the 6 by 12-in. control

*Reinforced Concrete Building Regulations and Specifications, *Proceedings*, American Concrete Institute, Vol. 24 (1928), Sec. 601, page 802.

†Standard Specifications for Concrete and Reinforced Concrete, *Proceedings*, American Concrete Institute, Vol. 21 (1925), Sec. 103, page 366.

cylinders are also given in the same table. A comparison of values given in this table shows that the computed compressive stresses in the beams at maximum load were much greater than the cylinder strengths, the excess over the cylinder strength ranging from 27 to 141 per cent. These results are in agreement with those of the earlier tests shown in Fig. 1 and 2.

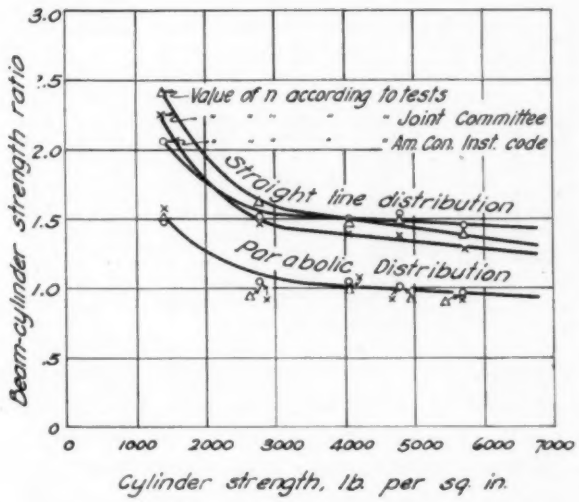

FIG. 21—RELATION OF BEAM-CYLINDER STRENGTH RATIO TO STRENGTH OF CONCRETE

Fig. 21 shows the beam-cylinder strength ratio, that is, the ratio of the computed beam stresses to the cylinder strengths for the different values of n used. It is noted from this figure that the ratios were much larger for concrete of low strength than for high-strength concrete. For concrete strength of more than 3000 lb. per sq. in., however, the decrease in the ratio with the increase in strength was very small. When the values of n used in the computations were in accordance with the American Concrete Institute code the beam-cylinder strength ratio was about 2.0 for 1500 lb. per sq. in. concrete, 1.75 for 2000, 1.55 for 3000, 1.50 for 4000, 1.45 for 5000 and 1.40 for 6000 lb. per sq. in. concrete. For the two other values of n used the relation between

the beam-cylinder strength ratio and the strength of the concrete follows approximately the same trend as the one found using n according to the American Concrete Institute code.

The differences in the ratios for the three different sets of values of n were not large.

TABLE 7—BEAM-CYLINDER STRENGTH RATIO USING PARABOLIC STRESS DISTRIBUTION, $r = 2$

Group No.	Depth d, In.	Cylinder Strength Lb. Per Sq. In.	Stress Computed By Parabolic Formula Using n According to			Beam-Cylinder Strength Ratio For n According to		
			A. C. I.	J. C.	Observations	A. C. I.	J. C.	Observations
1	10.2	1390	2050	2190	2090	1.47	1.58	1.50
2	10.3	2790	3030	2960	2860	1.09	1.06	1.03
3	10.3	4070	4380	4120	4000	1.08	1.01	.98
4	10.1	4800	4920	4770	4800	1.02	.99	1.00
5	10.2	5740	5580	5280	5440	.97	.92	.95
6	14.2	2590	3370	3360	3280	1.30	1.30	1.27
6A	14.1	4130	4380	4110	4020	1.06	1.00	.97
7	12.2	2950	3230	3250	3050	1.09	1.10	1.03
8	8.0	2760	3240	3190	3120	1.17	1.16	1.13
9	5.9	2900	3470	3380	3300	1.20	1.17	1.14
10	4.1	2820	2950	2890	2810	1.05	1.03	1.00
10A	4.1	3810	4170	3980	3920	1.09	1.05	1.03

Table 7 shows the extreme fiber stress in the beams at maximum load as computed by the parabolic formula, that is, the formula in which $r = 2$. The values of n used in the computations were the same as were used with the straight line formula. The compressive stresses were considerably less than those found for the straight-line distribution, but for cylinder strengths from 1400 up to 4000 lb. per sq. in. they were greater than the cylinder strengths. For higher cylinder strengths the computed stress in the beam was approximately equal to the cylinder strength.

The beam-cylinder strength ratios plotted in Fig. 21 for parabolic distribution were found to follow approximately the same trend for different strengths of concrete as for the straight-line distribution. The effect of differences in the method of determing n on the beam-cylinder strength ratio was found to be negligible for the stresses computed by the parabolic formula.

25. *Effect of Depth of Beam on Beam-Cylinder Strength Ratio.* The beam-cylinder strength ratios given in Tables 6 and 7, for beams with different depths, and two strengths of concrete (2800 and 4000 lb. per sq. in.) have been plotted in Fig. 22. The results for the concrete having a strength of 2800 lb. per sq. in. give some

indication of a slight increase in the ratio with increase in the depth of the beams. However, a curve drawn so as best to fit the points, indicates a higher beam-cylinder-strength ratio for the 10-in. depth than that given in Fig. 21 for the 10-in. beams made with concrete having a strength of 2800 lb. per sq. in. This throws some doubt upon the justification for concluding that there was an increase in the beam-cylinder-strength ratio with an increase in depth of beam. This doubt is emphasized by the

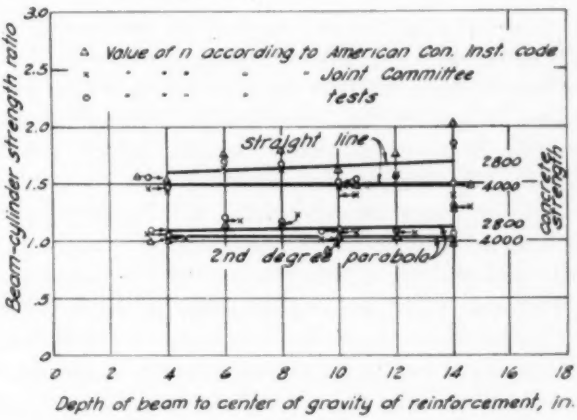

FIG. 22—RELATION OF BEAM-CYLINDER STRENGTH RATIO TO DEPTH OF BEAM

fact that the ratios for the 4000-lb. concrete showed no such increase in the beam-cylinder-strength ratio with the increased depth of the beam. From the data obtained, the most logical deduction seems to be that the beam-cylinder-strength ratio was independent of the depth of the beams.

26. *Shearing and Diagonal Tension Stresses.* Table 5 shows the shearing stresses in the beams developed at maximum load. These shearing stresses were computed by the formula

$v = \frac{W}{2jbd}$ where

v = shearing stresses and

W = total load on the beam

The value of j used in this formula was computed from the straight-line relation between stresses and strains, and the value of n was that found in the tests. On account of the fact that the strengths of the concrete were considerably higher than the strengths assumed in the design, the shearing stresses were correspondingly higher. However, none of the 10-in. beams failed because of high shearing stresses, nor were diagonal tension cracks found in any of these 10-in. beams. All of the beams having a depth to the center of gravity of the longitudinal reinforcement of 12 and 14 in. respectively, showed diagonal tension cracks in the web at a load equal to, or less than the maximum. Only one of these beams, however, failed in diagonal tension. This was the one in which 11-in. stirrups were attached to the lower layer of bars. The stirrups extended to about half way from the neutral axis to the top of the beam, that is, to about 4½ in. below the top of the beam. At the maximum load a crack extended nearly horizontally at about the elevation of the upper ends of the stirrups, indicating that the stirrups were too short to be fully effective. Anticipating difficulties due to the shortness of these stirrups the other two beams of the same group had the stirrups attached to the upper, instead of the lower, layer of longitudinal bars and the upper and lower layers were connected by means of spacers welded to both layers. This raised the tops of the stirrups 1¾ in., or to within 2¾ in. of the top of the beam. Although diagonal tension cracks were developed in these beams, the beams did not fail in diagonal tension. These facts bring out well the importance of extending the stirrups from the reinforcement to as close to the compression surface of the beams as possible. Evidently extending to a point half way between the neutral axis and top of beam was inadequate, and extending 1¾ in. higher was sufficient to prevent failure by diagonal tension, but probably extending it still higher would have been better. Fig. 11 is a photograph of Beam A of Group 6 which failed in diagonal tension.

Only one of the beams, 8B, shallower than 10 in. (from compression surface to center of gravity of steel) developed diagonal cracks, and this one failed suddenly in diagonal tension as soon as the diagonal cracks appeared. However, this beam carried

approximately the same load as the average of the other two beams of the same group. The average shearing stress for these beams, 8A, 8B, and 8C, was about 280 lb. per sq. in. and the shearing stress assumed in the design was 267 lb. per sq. in. After the test, the concrete was cut away from around the inclined bar and a careful examination showed that there was no slipping of the bar, the welding was still intact, and the steel showed no scaling such as should be expected if it had been stressed beyond the yield point. Fig. 12b shows the photograph of the beam with the concrete chipped away to expose the inclined bar in the region of failure.

The reinforcement for web stresses proved adequate for the purpose of the tests. It was desired to develop compression failure in all the beams. Even though the compressive strength of the concrete and the shearing stresses in the beams at maximum load were higher than the values for which the beams were designed, only two beams failed by diagonal tension. Even in these cases the loads carried were so close to those carried by the companion specimens that the compressive stress in the concrete when failure occurred must have been almost equal to the highest that could have been developed.

27. *Stresses in Reinforcement.* In Fig. 23 the stresses determined from the strains observed at the level of the center of gravity of the reinforcement are shown as ordinates, and the stresses computed by the straight line formula using n as specified by the American Concrete Institute's code, as abscissas. The plotted points represent the results from this series of tests and the solid straight lines are graphs of the equations given in the figure. These equations represent closely the relation between observed and computed stresses in an extensive series of tests carried out by the United States Geological Survey under the direction of Richard L. Humphrey about 25 years ago. On the whole the observed stresses agree very well with the computed stresses, rather more closely in fact than in those of the Geological Survey tests previously mentioned. For the high percentage of reinforcement used in these tests the straight line formula serves with sufficient accuracy for design purposes for computing the stresses in the reinforcement. It is not to be expected that agreement between observed and computed stresses will be as close

with low percentage of reinforcement. This is brought out very well in the paper* formerly referred to.

As illustrated by the beams of group 1, Fig. 23, plotted as triangles, the divergences of the observed stresses from the com-

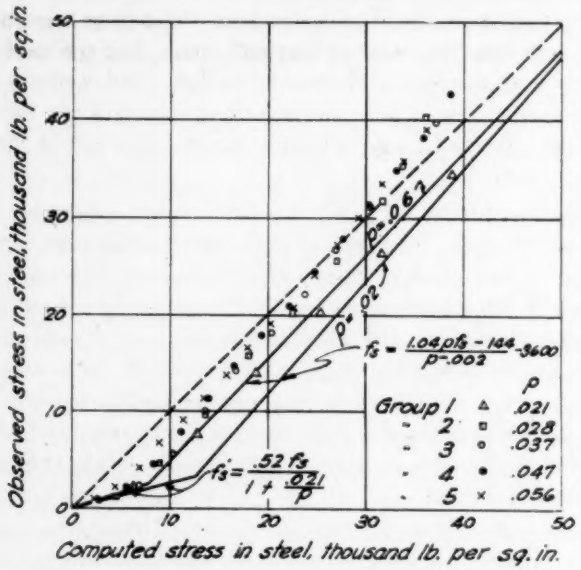

FIG. 23—RELATION BETWEEN OBSERVED STRESSES IN STEEL AND STRESSES COMPUTED ACCORDING TO A. C. I. SPECIFICATIONS

puted stresses was somewhat greater than for any of the other beams. The concrete in these beams had a strength of only 1400 lb. per sq. in., whereas the next highest strength was 2800 lb. per sq. in. However, the amount of reinforcement was not much lower for group 1 than for group 2. This disparity in the amount of reinforcement and the strength of the concrete between this and the other groups may account for a difference in relation between observed and computed stresses. Nevertheless, the difference has not been accounted for in detail.

28. *Stresses in Concrete.* Fig. 24 shows the relation between the so-called observed compressive stresses in the beams and the

*H. M. Westergaard and W. A. Slater, "Moment and Stresses in Slabs." *Proceedings*, American Concrete Institute, Vol. 17, page 480, 1921.

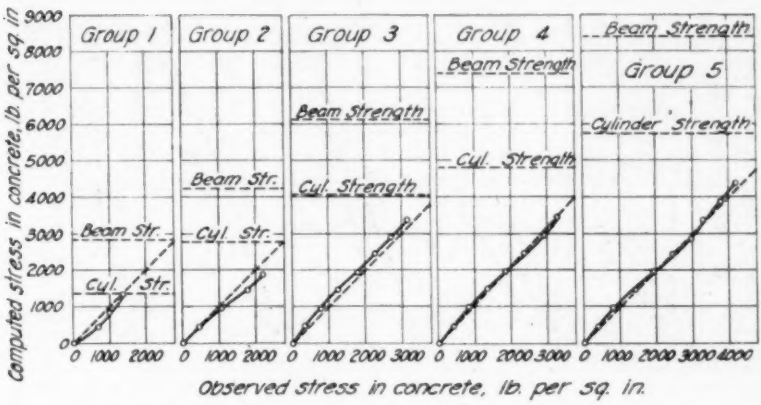

FIG. 24—RELATION BETWEEN OBSERVED STRESSES IN CONCRETE AND STRESSES COMPUTED ACCORDING TO A. C. I. SPECIFICATIONS

stresses computed by means of the straight line formula. The value of n used in these computations was according to the specification for the American Concrete Institute code. For the strains observed at the extreme fiber of the beam at different loads, corresponding stresses were taken from the stress-strain curve for the control cylinders. These are the so-called observed stresses for the beams and have been plotted as abscissas in Fig. 24. The ordinates in this figure are the computed stresses for the same loads on the beams. The dotted line gives the condition for equal computed and observed stresses. The stress-strain curves do not go as high as the strength of the cylinders because it was necessary to remove the measuring instruments before failure of the cylinder, in order to avoid damage to the instrument. The curves of the observed stresses therefore generally do not extend as high as the cylinder strength. They go far enough, however, to indicate that up to the stresses as high as the strength of the cylinders there is a close agreement between the observed and the computed stresses. The indication is that in a beam at the maximum load the compressive stresses are in agreement with the stresses for corresponding strains in the cylinders up to a height above the neutral axis at which the stress is approxi-

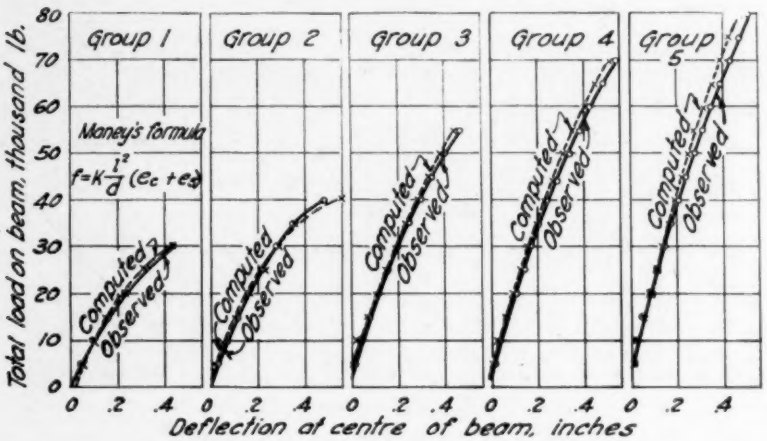

FIG. 25—RELATION BETWEEN OBSERVED DEFLECTIONS OF BEAMS AND DEFLECTIONS COMPUTED BY MANEY'S FORMULA

mately equal to the cylinder strength. For points above this height the stresses in the beam are not known, but computations indicated that they must be somewhat greater than the cylinder strengths in order to produce equilibrium between the internal and external moments. This finding is in conformity with the fact previously pointed out that the maximum stress found in the prism tests was about 8 to 20 per cent greater than the cylinder strengths for the same concrete.

29. Deflection. The average deflection for the different groups of beams of the five different strengths included in the series in which the size of the beams remained constant is shown in Fig. 25. For comparison deflections computed from a formula proposed by Prof. G. A. Maney* have also been shown. The solid curves represent the observed deflections, while the dotted curves represent the deflections computed by means of Maney's formula. Fig. 25 shows that a very good agreement exists between the computed and the observed deflections. This indicates that an intimate relation exists between the measured deformations of a reinforced concrete beam and the deflections. The deflection of reinforced concrete beams can therefore be computed as ac-

*G. A. Maney, "Relation between Deflection and Deformation in Reinforced Concrete Beams." *Proceedings* of the American Society for Testing Materials, page 310, 1914.

curately as can the deformations. It will also be noted from Fig. 25 that the total observed deflection near the maximum load was practically equal for all the beams included in this series.

TABLE 8—OBSERVATIONS NEAR MAXIMUM LOAD

Group No.	Maximum Load Lb.	Measurements at Last Observation					Computed jd, In.
		Load Lb.	Strains in Steel	Strains in Extreme Concrete Fiber	Deflection In.	kl, In.	
1	32,520	30,000	.00116	.00206	.44	6.4	8.37
2	44,710	40,000	.00136	.00214	.48	6.6	8.36
3	63,300	55,000	.00135	.00204	.46	6.1	8.25
4	74,830	70,000	.00145	.00250	.54	6.3	8.16
5	87,410	80,000	.00133	.00231	.51	6.4	8.14

In order for the deflections for all beams to be equal, Maney's formula shows that the sum of the unit deformations in the concrete and in the steel must be the same for all groups. Reference to Fig. 23 shows that the observed stresses in the reinforcement were approximately the same for all groups at maximum load, except possibly group 1. Therefore, the deformations in the concrete must also have been nearly the same at maximum load for all groups. That this is approximately true may be seen directly from Table 8.

If equality of the compressive strains and equality of the tensile strains for all groups existed at the maximum loads the arm, jd, of the resisting moment and therefore the value of j, must have been nearly equal for all groups. This also is shown in Table 8 to have been approximately true. It will be seen from the straight line formula for the position of the neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn$$

that the value of k, and therefore the value of j, depends entirely upon the product pn. In general, the value of p increased, and the value of n decreased as the strength of the concrete increased, maintaining the product pn nearly constant. The resulting computed value of j in these beams ranges only from .837 to .814. This seems to indicate that for design computations of stresses in the reinforcement of concrete beams the value of j might well be taken as constant regardless of the values of p and n.

SUMMARY

30. *Summary.* (1) In the tests carried out for this investigation all the beams had a width of 8 in. and a span of 9 ft. 6 in. The loads were applied at approximately the one-third points of the span.

(2) The percentage of longitudinal reinforcement varied from 2 to 6. This was sufficient to prevent any failures in tension in the reinforcement. The web reinforcement was designed to prevent failure in diagonal tension.

(3) Two series of tests were included. In one of them 5 different strengths of concrete were used, ranging from 1400 to 5800 lb. per sq. in. as shown by the tests of the 6 by 12-in. control cylinders. In this series the depth from the compression surface to the center of gravity of the reinforcement was 10 in. In the other series, varying depths were used, ranging from 4 to 14 in. In this series two strengths of concrete were used, 2800 and 4000 lb. per sq. in. respectively.

(4) In the mixes for all strengths in both series the fine and the coarse aggregates were 40 per cent and 60 per cent respectively of the total aggregates. The water content was maintained constant at 40 gal. per cu. yd. of the resulting concrete. This gave very nearly the same consistency for all mixes.

(5) In these mixes the quantity of cement ranged from 3.6 to 7.7 sacks per cu. yd. of concrete. The strength of the concrete increased in almost direct proportion to the increase in the cement content.

(6) A high degree of uniformity in the strength of the concrete was secured. This is indicated by the fact that the average of the maximum variations in the concrete strength for the different groups was only 6.4 per cent. The corresponding average of the maximum variations for the beams of the different groups was 8.2 per cent.

(7) All beams, except two, failed in compression of the concrete close to the center of the span. The two exceptions noted were beams which failed in diagonal tension. However, even these beams developed shearing stresses as high as those for which the web reinforcement was designed and the beams were close to failure by compression. The failure by diagonal tension

was due to the development of a higher compressive strength than was anticipated in the design.

(8) The 8 by 8 by 12-in. prisms loaded at the edge of the middle third of the cross-section developed compressive stresses about 20 per cent greater than the strength of the control cylinders from the same mix.

(9) The compressive strength of the concrete in the beams as computed by the straight line formula was about 25 per cent greater than the compressive stress developed in 8 by 8 by 12-in. prisms loaded at the edge of the middle third of the cross-section.

(10) The beam-cylinder strength ratio varied with the strength of the concrete. Using the straight line formula for computing stresses in the beams the ratio varied from more than 2.0 to 1.4 for concrete having a range in strength from 1400 to 5800 lb. per sq. in. Using parabolic stress distribution the beam-cylinder strength ratio varied from about 1.5 to .95 for concrete having a range in strength from 1400 to 5800 lb. per sq. in. For concrete having a strength above 3000 lb. per sq. in. the ratio decreased very little.

(11) Variations in the assumed values of n had a small effect on the beam-cylinder strength ratio for computations made by the straight line formula, and a smaller effect for computations made by the parabolic formula.

(12) Variations in the depths of the beams showed some effect on the beam-cylinder strength ratio. However, this effect was very small, and not entirely consistent. The data seemed to justify the conclusion that the effect of variation in the depths on the beam-cylinder strength ratio was negligible.

(13) The relations between strength of concrete determined by beam tests and by cylinder tests, found in this investigation confirmed in a general way the results reported by Emperger, Graf and Bach, and by Slater and Zipprodt.

(14) The observed stresses in the steel agreed fairly well with the stresses computed by the straight line formula.

(15) A very close agreement was found to exist between the observed compressive stresses in the concrete of the beams and the stresses computed by the straight line formula up to a stress equal to the strength of the control cylinders. At the top of the

beam the compressive stress seemed to be somewhat larger than the cylinder strength.

(16) The deflections observed near the maximum loads on beams of different strengths but equal dimensions were found to be nearly equal. The deflections observed at different loads agreed very well with the deflections at the same loads computed according to Maney's formula from the observed deformations. This is consistent with the fact that the arm *jd* of the resisting moment was found to be nearly constant for all the beams.

Readers are referred to the JOURNAL *for December 1930 (Proceedings Vol. 27) for discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1930.*

Discussion of Paper by I. E. Burks

"Concreting Methods at the Chute a Caron Dam"*

Convention Discussion

A. E. Lindau (Chicago)—I am not going to be deprived of a chance to talk, gentlemen, after all the effort I spent in trying to impersonate Mr. Burks. I am somewhat embarrassed in introducing Mr. Burks in his presence, because I intended to give a little background to his paper and I may not be as accurate in some of the things I state as I should like to be. However, that can all be rectified in the records and we can have all our facts straight. I am going to take you, for a moment, with me up to where Mr. Burks came from. I want to say that the location of this job, as nearly as I can get it from my observation and also from reading Mr. Burks' paper, is about as far from Ignoravia† as any place could be. I do not know just where Ignoravia is; if it is somewhere between Chicago and New York, I suppose it is Pittsburgh. The place where this work is going on is under the shadow of the Arctic Circle, if it has any shadow, on the outskirts of civilization, as it seemed to me, beyond which there wasn't anything but bush, and when Mr. Burks gets through with his presentation, I think you will agree that he has carried civilization up to the waste and wilderness of the northern part of our continent. My occasion for an opportunity to see this work came in connection with a committee assignment, Committee 801, Durability of Concrete, of which I happen to be

*Mr. Burks' paper having been published in the Journal for Feb. 1930 (A. C. I. Proceedings, Vol. 26, p. 315), the author was to have made a brief presentation of its outstanding features at the 26th annual convention. A few days before the meeting Mr. Burks wired he could not be present at New Orleans. The Program Committee drafted A. E. Lindau, who had been over the work at Chute a Caron, to understudy for Mr. Burks. When President Boyer called for Mr. Burks' paper and upon Mr. Lindau to present it, he was met on the way to the rostrum by Mr. Burks, who, changing his plans late, had slipped in unannounced. The situation caused a good deal of merriment and accounts for Mr. Lindau's participation in the discussion.—Editor.

†Referring to the mythical land of hap-hazard concreting methods, described by Arthur R. Lord in his convention presentation of the report of Committee 502—See March Journal (A. C. I. *Proceedings*, Vol. 26, p. 588.)

chairman, is engaged in a survey of existing concrete structures. It has done a good deal of work during the past year as a committee, and a great deal more work has been done by the individual members of the committee. Mr. Viens, a member of the committee, arranged a trip into Canada for his committee, which is of a similar character, of the American Society for Testing Materials, and invited Committee 801 to join him in a survey of concrete structures in Canada. We saw a good many structures, principally hydraulic structures and those of you who have given any time or thought to the production of concrete for hydraulic structures, probably realize what a problem is presented to the engineer and constructor in order to get a job that will last, that will be permanent and satisfactory as a structure. Before arriving at the site of this work that is being done by Mr. Burks, we saw several hydraulic structures, particularly dams, some of them of the type which may be designated as comparatively thin slabs supported by walls, in which the water was very evidently getting in its work. Others were of the gravity type, in which the water, the frost and the action of the air was also getting in its work. It was therefore a very great privilege for us to take a trip to the outpost of civilization. I do not know just exactly how far north of Quebec it is; it is up in the Lake St. John region, a night's ride. We landed there in the morning, at Jonquiere.

As you know by the paper, the dam is at Kenogami, near Chicotimi. Chicotimi is on the Saguenay river, an Indian word which means "deep up to here," the Saguenay being navigable up to that point. We found one of the largest concrete construction jobs probably on the continent. We found men in charge of this job who, apparently, have absorbed the information which comes to us through the Institute in the production of concrete. We found a job organized, it seemed to me, to the last detail of perfection, with a laboratory and the men in charge of the laboratory, who were continually controlling the quality of concrete, who were controlling the mixture of the aggregates and the result of that very highly efficient organization is probably going to produce a concrete job upon the Saguenay river that Committee 801 can visit in five or ten or twenty years—if it is longer than that I won't be there—and no doubt have every

reason to feel that it is going to be satisfactory. So, with that as a little introduction and a little background for the work that Mr. Burks has done, I am very pleased to yield the floor to the man who is in charge and can tell you more about it.

I. E. Burks (Kenogami, Quebec)—It was twenty-six below zero when I left camp four days ago, and the warmth of this introduction, coupled with this balmy air, has almost made me perspire. (Mr. Burks summarized his published paper briefly.)

Duff A. Abrams (New York City)—I would like to ask Mr. Burks a little more about this factor that he uses in reference to the mortar test cylinders as compared with the larger concrete specimens. I read the paper, but I am not sure that I got that just straight. Do you figure that the mortar cylinders are 95 percent of the concrete?

Mr. Burks—No, from 110 to 135 percent of the concrete. They are always higher.

Mr. Abrams—I wonder if there is another factor involved there that may not have been considered? As I visualize a concrete test, the first thing that happens is expansion of the diameter of the specimen; in other words, secondary stress. Concrete, in reality, deals with tension, so that the cylinder must be distorted to a certain extent, to give tension failure. Now, a small cylinder will reach a distortion of its diameter which will produce failure very much later than a large one; consequently I would expect just that result, that a small cylinder will give higher distortion than a large one; in other words, with a smaller cylinder you would get higher value with exactly the same materials.

Mr. Burks—That is actually what happens. Our 2 by 4-in. cylinders show higher strength than the 6 by 12-in. and 8 by 16-in. specimens, though all are out of the same mix.

Mr. Lindau—I think we have one of the most interesting and valuable papers that has ever been before you. I think that while we have Mr. Burks here and he is still warm, in this warm climate, that we ought to get all we can out of him. I do not know how many of you have read the paper; as the President said, I had to read it. In addition to my experience up there, I have a number of things in my mind about it. Some of the things struck me as being interesting, at least they were new to me.

Maybe I am one of these office men who do not go out on the job often enough, but so far as I know, I have not been on a concrete job where they have operated their mixer with a flashlight to tell them when to start their mixer and when to turn it off and how to handle all the various operations with a sort of a traffic signal system. I have not been on any other job where the concrete came to the forms in trainloads and they had a dispatching system to handle the cars. That is rather a new idea in the transportation of concrete, at least to me; and then there is another picture that sticks in my mind about that job. I remember in the discussion some years ago of concreting methods at the Muscle Shoals dam, John Earley was struck by the remark made by a man describing the concrete, that they measured its workability by the depth that the men sank into the concrete, whether up to their ankles or part way to their ankles—that that was a very simple and easy way to tell what kind of concrete you had. I climbed up on the forms and looked over the edge where they were putting in the concrete on this job on the Saguenay river, and the men were not sinking up to their ankles or part way to their ankles, they seemed to be slipping around on a material that had the consistency of blue clay that has a little water in it. There was not very much appearance of any water anywhere on the job. While these men were at work, a batch of concrete would be dumped, and two men would put the little vibrator on it, and the concrete would melt down about them. Those are some of the features in connection with this job that were quite a novelty to me and will be to most of you here, probably, who have heard the paper.

Arthur R. Levison (Pittsburgh, Pa.)—I would like to ask the maximum period from the time a batch goes to the mixer until it is deposited in the work? From the reading of Mr. Burks' paper, I would assume it to be considerable, owing to time in transit, and delay at point of deposit. I would like to know the condition of the concrete that was held the maximum time as determined visually or by tests, or both.

Mr. Burks—The extreme haul is about four miles, and the maximum age of concrete on this haul would be about fifty minutes when it is placed in the forms. We have a one hour law;

if concrete is over an hour old we throw it away. The condition of concrete of questionable age is determined visually and not by any test. Apparently there is very little difference in workability in concrete fifty minutes old and one hour old.

S. C. Hollister (Swarthmore, Pa.)—I notice the statement is made that when the vibrators are used, a stiff mix is used to the extent that the batch yield can be increased five to ten percent. I would like to know on what basis the batch yield may be defined in this case?

Mr. Burks—We meant that without the vibrators the concrete would have to be of such consistency that it could be booted in place by the ordinary methods, but where you use vibrators you can add more sand and stone to a given quantity of cement and water and increase your batch yield.

Mr. Hollister—On what is the percentage based?

Mr. Burks—On the percentage of the yield you would get if the workability was being regulated to fit ordinary placing methods where the concrete would have to be softer than if you were using vibrators.

Mr. Hollister—You mean the percentage of volume in place?

Mr. Burks—Yes sir. For example, a given quantity of cement and water might take only enough sand and stone to produce three and eight-tenths yards of *soft* concrete; while if you could use *stiff* concrete you could add enough *more* sand and rock to your batch to make it yield say four and two-tenths yards of concrete.

Mr. Hollister—Using the same proportions otherwise used?

Mr. Burks—The quantity of cement and water would be the same but the quantity of sand and stone would be greater, so the proportions would be changed somewhat.

Ernest Ashton (Allentown, Pa.)—I would like to ask whether the interpretation on workability of the mix was given by the man at the mixers or the man placing the concrete, and secondly, whether the interpretation of the quality of the concrete is based on compression test specimens?

Mr. Burks—The workability is regulated by the inspector at the form who is in contact with the mixing plant inspector by

telephone. The inherent quality of the concrete is based on the compression test—but we take pains to see that that quality is not lowered by improper placing in the forms.

Alexander Allaire (Atlanta, Ga.)—I did not quite catch whether with the electric vibrator, it is not necessary to spade the forms in the face of the concrete. Second, is there any trouble in getting the concrete out of the bucket after it had been in 50 minutes?

Mr. Burks—We have two men with each placing gang to spade the form faces. There is no trouble in getting the concrete out of the buckets because we use a bucket designed especially for stiff concrete—it has vertical sides and is center-dump. It only requires about fifteen seconds for a bucket to actually discharge its load.

R. L. Leftwich (New York City)—We have used the electric vibrator on a job for placing concrete in pre-cast piles. The specifications permitted driving the pile when a strength of 3500 lb. per sq. in. was attained. The vibrators helped to obtain this strength at an average age of 12 days on account of reduction in water-cement ratio. A ½-in. to 1-in. slump concrete could be placed with the vibrator where spading would have required a 4-in. slump.

Since no additional cement was required to obtain the specified strength at an earlier age considerable economy was effected. There is also a saving in labor by using the vibrator as one man with a vibrator can easily do the work of four or five spaders.

Drying up concrete to a consistency best for placing with vibrators may according to some authorities reduce the spread of mortar which produces that skin protection so desirable in concrete around sea water. We are now making some tests to determine the facts on this point.

Lloyd Wilson (Cleveland, O.)—I would like to ask Mr. Burks, if at twenty-six degrees below zero the day he left, they were concreting that day? If not, at what temperatures they stop concreting, and at what temperature they put the concrete into the form?

Mr. Burks—We did not happen to be concreting on the day I left camp, but we placed concrete during November when it was from 30 to 40 degrees below zero. We have no low temperature limit on when to stop placing. We heat the water, heat the sand, and the batches leave the plant at from 85 to 90 degrees. We sometimes lose up to 8 per cent of heat in delivering it to the forms—depending on the length of haul and weather conditions.

Mr. Wilson—Do you give it any protection after it is in the form?

Mr. Burks—Oh yes, that is all covered in detail in my paper.

C. P. Derleth (St. Louis)—I would like to ask Mr. Burks whether he made a report of any investigation as to the degree of compactness obtained in the elimination of air pockets?

Mr. Burks—I have never written any report on the degree of compactness obtained, but we have drilled out several large blocks of concrete from the dam and had them sawed into sections. From visual inspection we found what I would say were very few air pockets.

Louis J. Street (Winnipeg, Manitoba)—I was very much interested in Mr. Burks' statement as to 26° below zero. I would like to ask Mr. Burks if he ever tested any cylinders when exposed to zero temperatures over night? We have made experiments in Canada with concrete cylinders three weeks old and one week old, left out over night and tested the next morning, and we had double the strength that the companion cylinder would have. I do not know the reason for this, but it is a fact that concrete tested at zero weather or at a temperature below zero in that stage has doubled the normal strength.

Member—Frozen concrete.

Mr. Street—I would not call it frozen, but it is a phenomenon that I have not had an explanation of, but it is a fact.

Member—In regard to the question asked by the last speaker, a recent bulletin from the Iowa State Highway Commission reports the results of compression and beam tests made at different temperatures. In one test the temperature was 125; in some, at 70, in some at 32 and in some at 40 below zero. As the temperature was increased, there was an increase in strength

in each case, and the frozen cylinders and frozen beams gave about twice the strength, or 175 to 200 per cent of the specimens tested at 70°. The explanation given by Mr. Schlick was that the moisture in the specimen was frozen at these low temperatures and you were really testing ice. Professor Hatt has pointed out the same thing, that a beam when tested in a frozen condition gave seventy-five percent higher strength than when tested at a normal temperature.

Raymond E. Davis (Berkeley, Cal.)—In a series of tests at the University of California of concrete, previously hardened at various temperatures and then tested at those particular temperatures, we definitely established the fact that the higher the temperature the lower the strength, beginning way below freezing and going way up to the boiling point of water. There was a very pronounced increase of strength as the temperature was lowered, and we also discovered that the leaner the mix the less difference it makes. In the rich mixes, there was a material difference between the 70° concrete and the 100° concrete in the strength of the concrete at the time it was tested.

F. E. Richart (Urbana, Ill.)—I call your attention to a bulletin of the University of Texas, where in reference to tests made by Prof. Thomas Thayer, the same thing was mentioned. These tests showed that for the range of 200 degrees, that the increase in temperature produced a decrease of strength. That was particularly interesting to me, not only in the question of freezing, but that these cylinders above the normal temperature of 70°, showed that strengths decreased in proportion to the increase in temperature.

William S. Thompson (Chicago)—I would like to ask Professor Richart what was the age of these cylinders at the time they were subjected to these various temperature tests?

Mr. Richart—I think they were 28-day cylinders, and these temperatures applied immediately before testing.

C. M. Chapman (New York)—It seems to me that there ought to be something said here about this question that is under discussion, to make it very plain that it is not an advantage to cure concrete at these low temperatures. I think it is plain to those who have followed any of these previous tests, that all of

these specimens that showed the high strength at the low temperature, were cured and hardened under normal conditions that would promote hardening, and this high strength at low temperatures is not a result of curing at these temperatures.

Arthur A. Levison (Pittsburgh, Pa.)—In the northern states, Mr. Burks, we have experienced considerable difficulty in controlling the water cement ratio in concrete mixes when using live steam for heating the aggregates or introducing live steam into the drum of the mixer for heating the mixture. This comes about through the condensation of the steam coming in contact with the colder material. Is that a duplicate of your experience, and if so, how have you handled it in connection with the control of the water-cement ratio?

Mr. Burks—We make hourly tests to determine the moisture content in the sand and the amount of mixing water is reduced accordingly. According to tests we have made, the one-inch steam jets in the mixer add about six pounds of water per mixing minute when the boiler pressure is from 80 to 85 pounds.

BY HERBERT J. GILKEY* (BY LETTER)

SUCH papers as this one go far toward bridging the almost inevitable gap between laboratory findings and job applications. It is obvious that Mr. Burks makes his job his laboratory and refuses to permit mere yardage to suppress his inquisitiveness about things that may affect quality

While the whole paper testifies to the truth of the foregoing, the writer will confine his remarks to but one phase of Mr. Burks' practical investigational activity.

Over a period of several years the writer has found that mortars are invariably and fundamentally stronger than their concretes. Within the writers experience the percentage excess has ranged from 15 to more than 50 per cent and the differential is present for all ages, curing conditions, and a great variety of mixtures. A published statement[1] of these findings a few years ago created some little controversy on the subject.[2] There seemed to be a

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[1]"The Coarse Aggregate in the Concrete as a Field for Needed Research," *Proc.* A. C. I., Vol. 23 (1927), p. 363.

[2]Same, pp. 389-414.

distinct conflict of evidence on the part of different investigators. For this reason the writer has continued his researches along these lines and has followed similar work on the part of others with a great deal of interest. He has therefore known something of Mr. Burks work on this subject. A preliminary report of the Santeetlah tests appeared in the Technical News Bulletin of the Bureau of Standards.[3] This report was incomplete and apparently no other has appeared. It is extremely interesting therefore to see that Mr. Burks still feels that there is an appreciable differential between the strength of the concrete and its mortar.

Since publishing the original paper the writer has conducted several additional series of tests to supplement his previous ones and the evidence continues to give unqualified support to former findings. For years it has been his practice to weigh the mold plus specimen as made and again a few hours later prior to capping and after all water loss by leakage and evaporation had ceased. While thin mortars sometimes lose a higher percentage of their mixing water than do concretes (thus giving a relative lowering of the water-cement ratio), the strength differential from this source is but a fraction of the total. The water-cement ratio curve for the mortar is always above that for the concrete and this is easily recognized even though the over-lap be but partial on account of the relative thinness of the mortar.[4,5]

Within the past two months a new series of concretes and mortars from crushed granite and granite screenings shows the greatest differential yet observed. After correcting for leakage from the molds, 28-day concretes 1:2½:4 (by weight), of water-cement ratios of 0.90 and 1.00 respectively gave strengths of 3,300 and 2,600 lb. per sq. in. at 28 days while the 1:2.5 mortars gave 4,550 and 4,000 lb. per sq. in. Strength ratios are from 1.37 to 1.54. Seven-day tests gave parallel indications. Similar results were obtained when 1:3:5 concrete was compared with 1:3 mortar, all made from the same materials.

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[3]Jan. 1928, p. 5.
[4]*Proc.* A. C. I., Vol. 24 (1928), p. 157.
[5]*Proc.* A. C. I., Vol. 23 (1927), pp. 376, 377.

As the maximum size of aggregate increases it is possible to crowd more stone into mixture thereby effecting distinct economies from the standpoint of yield. While the use of large individual plums has generally been found to be undesirable because of interference with other operations, there seems to be definite economy in passing cobbles (10 in. or less) directly through mixers designed to accommodate them. As Mr. Burks has pointed out, it is expensive and impracticable to test concrete that includes cobbles. If the cobbles do have an influence upon strength or other properties, it is the engineer's business to know of it. The possible strength differential is probably more than offset by the added economy with a sufficient margin to warrant an enrichment of the mixture sufficient to counteract the effect of the cobbles.

Granting the existence of a difference it is up to the concrete technician to devise adequate and accurate means of inspection and control. Mr. Burks has ingeniously used the mortar as his criterion of concrete quality. In many cases it is probable that the mortar would be too fluid and the use of concrete samples obtained through a 1½ or 2-in. screen might well prove to be a more satisfactory yardstick.

In the meantime the importance and use of cobble-concrete is increasing and the "yardstick," whether of mortar or concrete, needs to be further calibrated. A refined properly controlled test program to supplement that which Mr. Burks conducted in connection with the Santeetlah Dam is a present necessity to this end. The undertaking is not a small one. If specimens varying materially in size are used, different temperature conditions and possible shrinkage stress within the mass may enter as disturbing factors. For specimens differing greatly in height "water gain"[6] might also prove to be the source of another variable. In the conduct of such a program it would seem highly desirable that every test include accurate determinations of lateral and longitudinal strains since they contribute much toward a better understanding of what is taking place within the mass.[7]

[6]Water-gain and Allied Phenomena, *Engineering News Record*, Feb. 10, 1927, Vol. 98, No. 6, p. 242.

[7]Some very recent 7 and 28-day tests of concretes identical except that the coarse aggregate in one group was all below one-inch and in the other it was all above 1½ inch, showed a

With his demonstrated research ability and previous interest and experience with the problem, it would be a fortunate circumstance if Mr. Burks might himself undertake to supplement his previous findings. In the meantime much credit is due him for his contributions in the field of large-scale concrete construction and control.

Readers are referred to the JOURNAL *for Oct. 1930 (Proc. Vol. 27) for further discussion mhich may develop. Such discussion must be available before Sept. 1, 1930.*

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more rapid lateral swelling in the concrete which contained the larger aggregate. It was evidently the result of pronounced slipping along the larger aggregate surfaces.

There was also a pronounced difference in favor of the concrete containing the smaller size aggregate. This was 28 per cent for 1:2.5:4 mix with 7.5 gal. per bag and only 5 per cent for 1:3:5 mix with 9 gal. per bag. Indications at 7 and 28 days were in close agreement.

Discussion of Paper by H. F. Gonnerman

"Study of Methods of Curing Concrete"*

Convention Discussion

D. D. McGuire (Nashville, Tenn.)—It may not be amiss to tell you of another research project, of a field nature, in Tennessee where we used 25 different types of curing, each repeated four times, on seventeen miles of concrete highway. There are 128,000 breaks. The interesting thing about this particular project is the fact that it compares a research type of cure with a section of concrete pavement cured by a standard earth and water method directly across the pavement. Comparisons to be drawn from the research results and the standard method will be from a very practical standpoint. I think we are going to get some results somewhat different from those Mr. Gonnerman has reported, although I am not prepared yet to give the final data. This is a cooperative project between the Bureau of Public Roads and the Tennessee State Highway Dept. Data will probably be published in *Public Roads*.

Ben Moreell (Washington, D. C.)—I think you might be interested in a survey made by the Navy Department, along the line Mr. Gonnerman described, except that we made no test. Our survey was intended to gather information from various practising engineers, our own among others, as to what results had been obtained with the different methods of curing concrete pavements. Now I think one of the points that Mr. Gonnerman touched on was not sufficiently emphasized. He mentioned briefly the effect of absorption of heat by the black surface coating. We found in our survey that the effectiveness of any particular kind of curing agent was dependent to a large extent upon climatic conditions. For example, the use of calcium

*Journal, A. C. I., Feb. 1930; *Proceedings*, Vol. 26, p. 359.

chloride as a surface treatment, in a hot, dry climate, will, we found, lead almost invariably to flaking and poor curing. Calcium chloride, as you know, is a deliquescent agent; it absorbs water from any source it can get it, and for that reason it is used to a large extent as a dust settler on earth and gravel roads. The action of curing in a concrete road is similar. In a dry climate, when it cannot get the moisture from the atmosphere, it takes it from the concrete. In any event, that opinion was confirmed by communicating with various practicing engineers. On the other hand, calcium chloride as a surface treatment, flake calcium chloride, has been very successful in humid atmospheres and it is quite easy to see why it should be. Usually a humid atmosphere is one where there is not a large daily variation in temperature. That brings up another point we found in our survey of the effectiveness of bituminous coatings—that where there is a large daily variation of temperature, there is considerable absorption of heat by the black coatings, and that the concrete is called upon to resist temperature variations at the very period in its life when it is not able to do so. In other words, we lose the insulating blanket that is obtained by the use of earth or straw curing, and I think that is a very important point. Our survey indicated that in such climates the use of a bituminous coating for curing resulted in appreciably increased cracking of the roads.

I was very glad to hear Mr. Gonnerman say that he does not consider calcium chloride used as an admixture, as a curing agent. Our investigation indicated that calcium chloride is purely and simply an accelerator, and if you look on it as such and provide sufficient water for water curing in the early days of the calcium chloride admixture curing, you will probably get something for your money; otherwise I am afraid you won't. I think the most important conclusion that we drew was the fact that in a hot climate—and we usually build concrete roads when the sun is boiling down—the effect of the insulating blanket obtained by wet earth or straw is something which should not be neglected.

Mr. McGuire—May I ask the last gentleman who spoke what he recommends for curing concrete pavements?

Mr. Moreell—I am glad to tell you; water curing. Now the people who try to sell you various materials for curing concrete pavements finally wind up and say "But there is the human

element; when you wet your earth covering or your straw covering, how do you know it is going to stay wet?" Why, hell, that is what we engineers are paid for, to see that it stays wet.

Mr. McGuire—I want to pass one bit of information that is not perhaps included in our cooperative research: there are five curing agents, we found, that are better than earth and water.

Mr. Moreell—I think that is very true; you may have found that, but don't forget the point I mentioned first, that the effectiveness of any curing agent depends almost entirely upon the climatic conditions. We found at Puget Sound, that calcium chloride surface treatment was very effective; we could not tell the difference between that and water curing, and it was cheaper for those particular conditions. But why? Because Puget Sound happens to be where the atmosphere is very, very humid. Now I am familiar with the experiments being conducted in Tennessee, and they are admirable; we will get a lot out of them; but not everything, and the reason is because the climatic conditions are not sufficiently varied there to bring out the difference in the results from that cause.

Mr. McGuire—There is a problem in curing that I think should not be missed. We sometimes have to lay concrete pavements where water is not available. In Tennessee we have a section where it is hard to get enough water for mixing, to say nothing about curing, and you can bet your bottom dollar that when contractors are laying concrete in that part of the country, they are not going to put any of the water in curing if they can help it, because they want it to lay concrete. There has to be some substitute that we can use for curing concrete pavements, or, as highway engineers, we are going to be faced with a great problem.

R. S. Hale (Montgomery, Ala.): I want to ask Mr. Gonnerman if he found any difference in the effectiveness of the different cures of slags, gravel, or stone concrete, and what he based his tests on? What aggregates did you use?

Mr. Gonnerman—We used only the one aggregate and one mixture—1:2:3 mix (dry rodded volume) concrete containing about six sacks of cement per cubic yard.

Mr. Hale—There is a field for investigation there; I believe there is some difference in the effectiveness of cures on different

aggregates, sand, gravel and limestone. I would like to hear some discussion on that.

Chairman Lord—Mr. Gonnerman, what was the water cement ratio?

Mr. Gonnerman—A little over five gallons per sack. I think the point the gentleman just made here has to do with the effect of the absorption of aggregates on the curing. He has the idea that by having an absorbent aggregate it takes quite a little water in the mixture which is later available for the hydration of the cement. We have not covered that point in our investigation. It might be well to go into it.

Chairman Lord—One of the interesting things to me in this paper is to compare these figures as you have them on pages 370 and 371. It is rather interesting to see the very great difference between that 7000-lb. concrete for water curing and the results found by some of the other methods.

BY W. W. ZASS*

WHILE curing is considered essential in the proper development of the qualities of any type of concrete structure it is probably practiced more extensively in connection with paving work than in connection with bridge or building work. This condition is doubtless due first to the greater area of surface exposed without protection of form-work and the comparative ease and low cost involved in comparison to applying the same methods to structural work.

In connection with paving work it is common practice to specify a curing period ranging from 7 to 14 days and opening the pavement to traffic at the end of that period or to specify a predetermined strength requirement and open the pavement to traffic as soon as such strength is obtained regardless of the curing period specified. With this condition in mind it is believed that a comparison of results obtained from different curing agents should be evaluated to a curing period of a duration similar to which the structure is to be subjected rather than to a theoretical curing period of a duration that never obtains in actual practice. If a pavement is opened to traffic at the expiration of a 7-day or a 10-day curing period it is desirable to employ a curing agent that

*Engineer of Construction, Arkansas State Highway Commission, Little Rock.

FIG. 1

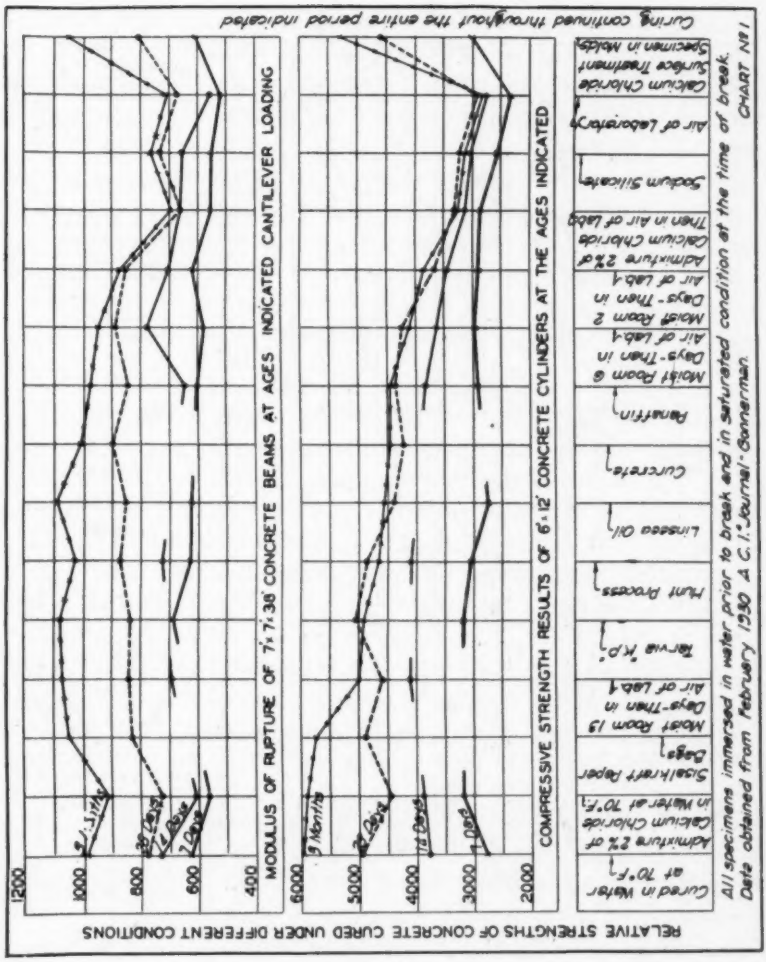
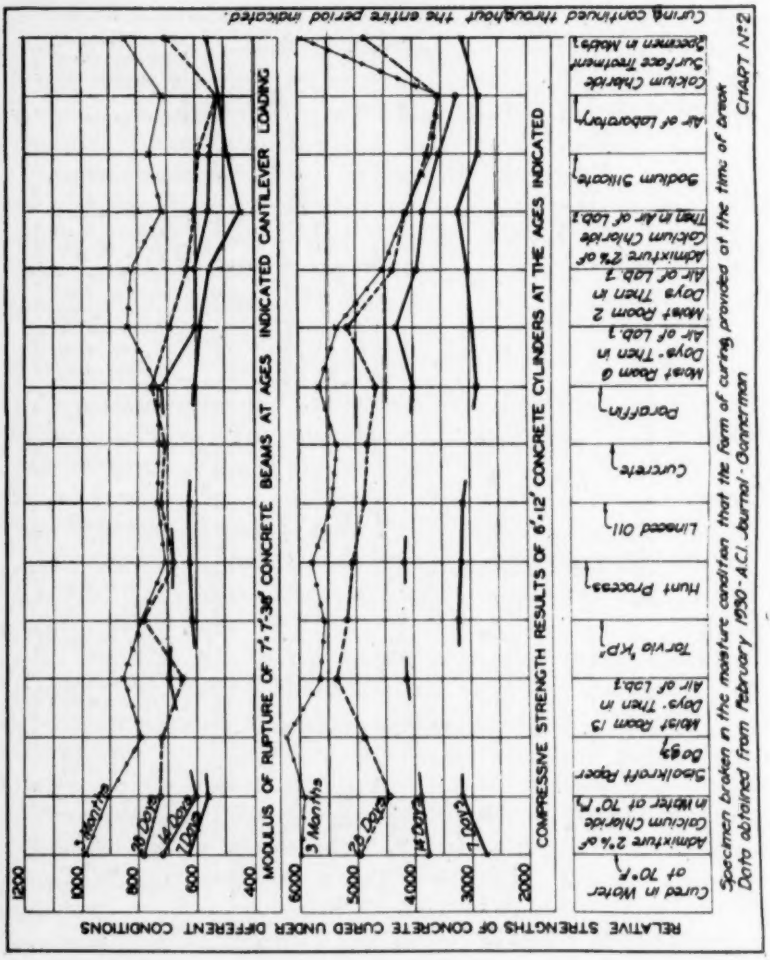

RELATIVE STRENGTHS OF CONCRETE CURED UNDER DIFFERENT CONDITIONS
COMPRESSIVE STRENGTH RESULTS OF 6"x12" CONCRETE CYLINDERS AT THE AGES INDICATED
MODULUS OF RUPTURE OF 7"x7"x36" CONCRETE BEAMS AT AGES INDICATED CANTILEVER LOADING
Curing continued throughout the entire period indicated
Cured in Water at 70°F.
Admixture 2% of Calcium Chloride in Water at 70°F.
Sisalkraft Paper
Moist Room 13 Days-Then in Air of Lab.
Ransome K.P.
Mat Process
Linseed Oil
Concrete
Paraffin
Moist Room 6 Days-Then in Air of Lab.
Moist Room 2 Days-Then in Air of Lab.
Admixture 2% of Calcium Chloride Then in Air of Lab.
Sodium Silicate
Air of Laboratory
Calcium Chloride Surface Treatment Specimen in Molds
All specimens immersed in water prior to break and in saturated condition at the time of break.
Data obtained from February 1930 A.C.I. Journal-Sonnerman.
CHART No. 1

FIG. 2


RELATIVE STRENGTHS OF CONCRETE CURED UNDER DIFFERENT CONDITIONS
MODULUS OF RUPTURE OF 7"·7"·36" CONCRETE BEAMS AT AGES INDICATED CANTILEVER LOADING
1200
1000
800
600
400
3 Months
28 Days
14 Days
7 Days
COMPRESSIVE STRENGTH RESULTS OF 6"·12" CONCRETE CYLINDERS AT THE AGES INDICATED
6000
5000
4000
3000
2000
3 Months
28 Days
14 Days
7 Days
Cured in Water at 70° F.
Admixture 2% of Calcium Chloride in Water at 70° F.
Steelcroft Paper No 83
Moist Room 13 Days. Then in Air of Lab.
Tarvia "KP"
Hunt Process
Linseed Oil
Concrete
Paraffin
Moist Room 6 Days - Then in Air of Lab.
Moist Room 2 Days Then in Air of Lab. 7
Admixture 2% of Calcium Chloride Then in Air of Lab.
Sodium Silicate
Air of Laboratory
Calcium Chloride Surface Treatment 2 Decrem in Moist
Specimen broken in the moisture condition that the form of curing provided at the time of break
Data obtained from February 1930 - A.C.I. Journal - Gonnerman
CHART N°2
Curing continued throughout the entire period indicated.

will presage the greatest strength at that age regardless of what evaluation may be obtained at a 90-day or 360-day period if curing had been prolonged throughout that entire period.

The tests summarized in the "Study of Methods of Curing Concrete" by H. F. Gonnerman, do not emphasize the value of the results obtained through the use of the several curing agents at the 7-day and 14-day periods, with relation to the value obtained through water curing at the same periods, although this comparison is emphasized for the longer periods of 3 months and 1 year, which latter periods never obtain in practice. The data for such an evaluation is available however in the text of the article in question and the results are shown in both Fig. 1 and Fig. 2, together with the comparison at the age of 28 days, three months and one year. It is unfortunate that 7-day and 14-day results are not available for each type of curing as it is believed that it is on strength at this age that the evaluation of the several types of curing should be made.

An analysis of these charts indicates that neither concrete cured in water at a temperature of 70° F. nor concrete cured in a moist closet for 13 days gave a greater efficiency at the 7-, 14-, and 28-day periods than several other types of curing agents either in compressive strength or modulus of rupture. A review of the text indicates but a slight difference in depth of wear. From the standpoint of ease of application, positive control and economy in cost several curing agents now on the market excel any of the methods of water curing. It has been heretofore contended that some form of water curing is superior to commercial agents but the results of these tests do not support this contention for the length of curing period to which pavement work is usually subjected.

If, as it has been commonly accepted, a 7-day to 14-day curing period is satisfactory for concrete slab work, it should be equally satisfactory to use a commercial agent for such curing in accordance with the results of the study indicated, and recommendations to this effect should be emphasized in that such agents receive equal evaluation in comparison to some form of water curing.

BY R. F. REMLER*

WE appreciate the opportunity afforded us to discuss Mr. Gonnerman's paper.

Laboratory tests, as reported, should give at least relative results. However, the tests as conducted are not comparable to road construction, neither are they relative in many instances to each other. This is regrettable, as such papers are quite often referred to in sales promotion, and are unjust to those concerned.

A laboratory test of this nature, regardless of how comprehensive, is only a poor criterion by which to judge the practicability of a curing method for concrete roads. The writer knows of no extensive laboratory test on curing which has taken into account the drying effect of the wind and the radiated heat of the sun. The former accelerates the drying of the green concrete as much as any other factor, and the latter produces a different volume change where there are surfaces cured by various methods.

These various methods of curing depend on their different and individual properties, which react dissimilarly under laboratory and field tests. Conditions in the laboratory, which favor one method have quite the opposite effect on another.

Silicate of soda curing depends upon the setting of a gelatinous film, and the greater the drying elements, the more rapidly this film is produced. This is when curing is most needed and when silicate of soda is most effective. However, these are not the conditions found in laboratory tests. The average laboratory tests have given results with silicate of soda far below actual field tests, or cores from actual construction work. ("Special Investigation on Curing of Concrete Pavement Slabs," Proceedings Ninth Annual Meeting of the Highway Research Board, 1929.) For this reason, we do not recommend silicate of soda as a curing agent for interior work.

It is difficult to obtain a satisfactory film of silica of soda on a vertical surface, especially an oil-coated surface produced by oiled molds. In the paper to which we refer, the forms were taken from the silicate-of-soda-cured specimens and both the top and sides coated with silicate. In comparison, results are given on specimens cured in waterproof bags. This, one must

*Grasselli Chemical Co., Cleveland.

admit, produces an excellent moist cabinet, but we doubt the practicability of this procedure in road construction. Specimens, surface cured with calcium chloride, were left in the molds and the top edge dyked with paraffin to prevent moisture from running off. We have never seen the dyking of calcium-chloride cured roads, however, we feel sure it would be more necessary on a two-inch crown or grade than on level specimens.

The report shows, that cylinders not cured, but left in molds gave as good or better results than the various methods, with the exception of those immersed in water or placed in a moist cabinet until tested. Likewise, the beams tests on specimens not cured but kept in forms, show considerable increase over specimens kept in forms one day and uncovered in air the remaining time. This indicates that the forms played a considerable part in the water retention of these specimens. Therefore, results are not comparable, where one series is permitted to remain in the forms and the other taken out and the sides and tops coated. Where this is carried out on 6 x 12 in. cylinders, there is approximately ten times as much surface to be protected by a curing agent per unit volume as compared to cylinders left in forms, and treated only on the top surfaces.

The wear tests reported by Mr. Gonnerman show silicate of soda inferior to the bituminous methods of curing. This is not in accord with field tests made by the U. S. Bureau of Public roads and reported in the February 1930 issue of *Public Roads*, nor does it substantiate tests made in our own laboratory.

The absorption tests as conducted and reported are as incomparable as some of the other tests. It has been determined by experiment that any specimen of concrete washed with kerosene will repel water to the same degree, regardless of the curing method. Kerosene has a boiling point of approximately 150° to 300° C. and cannot be entirely removed by heating a specimen at 100° C. for a short time as stated in the paper.

Therefore, in view of these facts, we feel that Mr. Gonnerman's laboratory results are not relative in many instances to each other, nor comparable to actual road conditions.

BY JOHN A. FOCHT*

DURING the summer of 1929 the writer tabulated the results of compression tests made on concrete cylinders and cores taken from concrete highways laid in Texas in 1928. (This information is published in Bulletin 2922, Bureau of Engineering Research, University of Texas). This study included two jobs using various methods of curing, and the results may be of some interest at this time.

It must be remembered that the cylinders in the above study were made in the field, and cured in the field by the respective inspectors until the cylinders were from two to three weeks old, at which time they were shipped to the laboratory. In 1928, when these cylinders were tested, the treatment of the cylinders at the laboratory was as follows: The rough end or ends of the cylinder were smoothed with a hammer and chisel upon its arrival at the laboratory, and the end or ends capped with plaster of Paris. The cylinder was stored in air in the laboratory until it was twenty-eight days old, when it was crushed. The present practice is to cap the ends with neat cement after the ends of the cylinder have been smoothed, and then store the cylinder in the moist air closet for at least one week, or until it is twenty-eight days old, when it is tested in a moist condition.

Job number one covers a period from December, 1927 to June, 1928. The cylinders from December until about the first of May were reported as having been cured by the standard method, wet earth. Beginning about the first of May and continuing through the remainder of the job, about two-thirds of the cylinders were cured with sodium silicate, and one-third with wet earth. During the period when wet earth and sodium silicate were used as curing mediums, the average crushing strength of the wet-earth-cured cylinder was 5060 lbs. per sq. in., and the average crushing strength of the sodium-silicate-cured cylinder was 3570 lbs. per sq. in. No notes were found in the records indicating the type of curing of the pavement from which cores were taken.

On job two, part of the cylinders were cured with the Hunt Process and the remainder were cured in wet earth. During the

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first month on this job, April 1928, the Hunt Process was used entirely for curing, and then for several days in May, cylinders cured with the Hunt Process and companion cylinders cured with wet earth were sent to the Laboratory. After the first week in May all cylinders were cured in wet earth before being sent to the laboratory. The average compressive strength of the Hunt Process cured cylinder is 5300 lbs. per sq. in., and the average crushing strength of the wet earth cured cylinder is 6010 lbs. per sq. in. During the time when both methods of curing were

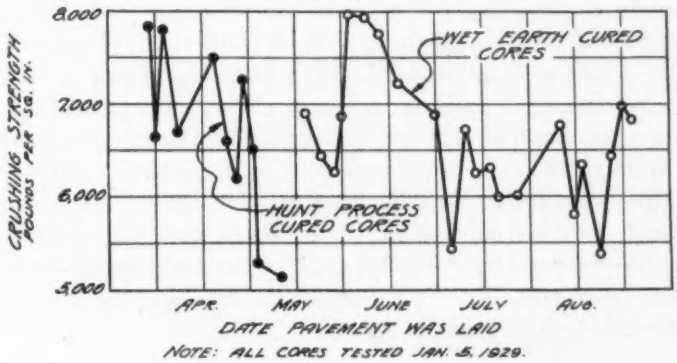

FIG. 1

used there was not a day when the highest Hunt Process-cured-cylinder was as strong as the lowest wet-earth-cured cylinder.

The pavement was cured with the Hunt Process from the beginning of the job in March until about May 10, according to notes accompanying the cores. From May 10 until the completion of the job the first of September, the notes indicate the pavement was cured with wet earth. The Hunt Process cured cores have an average crushing strength of 6680 lbs. per sq. in. at ages varying from 240 to 283 days. The wet earth cured cores have an average crushing strength of 6510 lbs. per sq. in. and the ages vary from 124 to 200 days. These cores were 6 in. in diameter, and about 6 in. high, as they were taken from a 6-in. pavement. In 1928 the treatment of the core was as follows: The core was measured for depth and diameter, and the area calculated upon its arrival at the laboratory. The end next to the subgrade was

smoothed by means of the hammer and chisel, the rough end capped with neat cement mortar, after which the core was placed in the moist air closet for one week to allow the cement cap to harden. At the end of this time the core was removed from the moist air closet, and a plaster of Paris cap put on both ends. The core was then stored in air in the laboratory for several days, usually from three to eight, and then crushed. The present practice in the laboratory is to cap both ends of the core with neat cement and as soon as the caps are sufficiently hard, it is stored in water for one week, and tested while wet. The crushing strengths of the cores given above have been corrected for the relation of h/d according to U. S. D. A. Bulletin 1216 page 35.

The accompanying curve has the crushing strengths of the cores as ordinates, and the date the pavement was laid as abscissas—the date affords an indication of the age, the oldest core is on the left and the youngest is on the right of the curve. All cores were tested on Jan. 5, 1929. Due to the variables in the field, weather conditions, temperature, etc., no conclusions were drawn from this curve other than the cores cured by both the wet-earth and Hunt Process show the tendency of an increase in strength with increase in age.

BY HERBERT J. GILKEY*

IN the midst of a concrete-minded age the confusion that has surrounded the curing and drying of concrete has been little short of amazing. While the efficacy of water as a curing agent has been recognized for many years, the importance of different moisture states at the time of test has been recognized but rarely and has theretofore been the source of much misleading inference from test results purporting to show the effects of different curing methods upon strength or other properties. The recent advent of a horde of patented curing agents that rival in number and claims the genus "admixture" has placed the whole subject of curing in the spot-light during the past few years.

Countless and costly curing investigations have been conducted. These have netted some information and a great deal of misinformation. Even among the better of these there have

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ULTIMATE STRENGTH VS. WET OR DRY CONDITION AT TEST

Test Age	Curing		Test Condition	Ultimate Strength			Flexure—Modulus of Rupture	
	Water	Air		Compression	Tension	Torsion		Series B[1]
Col. A	B	C	D	E	F	G	H	I
1 yr.	8.5 mo.	3.5 mo.	Dry	5090	495	665	850	856
1 yr.	8.5 mo.	3.5 mo.	Wet	3710	265	295	720	630
Ratio	(dry)/(wet)			1.37	1.87	2.25	1.18	1.36
1 yr.	3 mo.	9 mo.	Dry	4030	340	490	675	
1 yr.	3 mo.	9 mo.	Wet	3230	226	282	520	
Ratio				1.25	1.51	1.74	1.30	
1 yr.	28 da.	11 mo.	Dry	3540	332	463	624	
1 yr.	28 da.	11 mo.	Wet	2860	184	295	449	
Ratio				1.24	1.80	1.57	1.39	
1 yr.	7 da.	11.8 mo.	Dry	3080	297	418	632	
1 yr.	7 da.	11.8 mo.	Wet	2460	241	295	438	
Ratio				1.25	1.23	1.42	1.44	
1 yr.	2 da.	12 mo.	Dry	2470	248	394	540	530
1 yr.	2 da.	12 mo.	Wet	2030	226	273	416	294
Ratio				1.22	1.10	1.44	1.30	1.80
Size and No. Spec.				6 by 12 (1) 3 by 6 (2) 2 by 4 (2)	3 by 12 (2)	3 by 12 (1)	3 by 3 by 40 (1)	4 by 6 by 38 (1)
Kind Spec.				Cylinders	Cylinders	Cylinders	Beams	Beams
Span				—	—	—	38 in.	36 in.

Mixture 1:3.25 by wt. w/c = 1.00 (7.5 gal. per bag.)
Aggregate—Granite screenings 0-$\frac{3}{8}$.
Specimens tested wet were immersed in water 24 to 48 hours prior to test. There was practically no increase in weight after the first 24 hours.
(1) Beams of Series B (Col. 1) were made of same mixture 6 mo. prior to all other specimens. Total age was 1.2 yr. with 7 mo. water curing. Bottom pair were exposed to air at one day age.

usually been present one or both of the following disturbing factors to mask the true results of the tests.

(a) The moisture condition at test was not kept uniform and this supplied an extra and unevaluated variable.

(b) The tests were at such an early age that, drying and curing effects were intermingled and inseparable.

It was only in the best of the investigations that there was not present an added lack of control of such factors as accidental moisture changes via sub-grade, rain or fog and also uncontrolled temperature effects. The author has mentioned some of these items.

In the light of all these ill-conceived attempts to catalogue curing phenomena it is gratifying to have available at last the results from a *real curing investigation* planned and executed by one thoroughly familiar with the factors that heretofore have been so disturbing.

The evidence from these tests will be invaluable to highway departments and other concrete users in providing unbiased evidence for comparing and evaluating curing methods and agencies.

In the writer's work on curing* he has never investigated any of the patented curing compounds and the results of these tests are, to him, new and illuminating. His tests have overlapped those of Mr. Gonnerman in most other respects and have even gone beyond them in some regards. On all except one major point nothing but essential agreement is found.

On page 361 Mr. Gonnerman states: "In contrast to the cylinders, beams tested wet were generally stronger at all ages than beams tested 'as cured.' A few exceptions occurred in the 1-yr. tests."

In the writer's comparisons of beams, and compressive tensile and torsional cylinders, the saturated condition at test has consistently produced lower strengths than the dry or "as cured" condition. In other words the actions in compression, flexure,

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**Proc.* A. C. I., Vol. 22 (1926), pp. 395-431. *Proc.* A. S. T. M., Vol. 27 (1927), Part II, pp. 424-427. Trans. Am. Soc. C. E. Vol. 91 (1927), pp. 153-167, also *Proc.* Am. Soc. C. E., Jan. 1927, pp. 79-93. *Proc.* A. C. I., Vol. 24 (1928), pp. 238-239; 152-153; 758. *Proc.* A. C. I., Vol. 25 (1929), pp. 559; 546-549; 442-443. *Proc.* A. S. T. M., Vol. 29 (1929), Part II, pp. 705-706.

tension and torsion are all exactly similar. The writer can see no reason why all should not be affected the same way by moisture or by drying although it is test results and not opinions that must settle the question. At present the test results are not in agreement on this one point. Typical cases are shown in the table.

Not only do the writer's results fail to support Mr. Gonnerman's statement regarding beams but tests of Professor Hatt[1] indicate that saturated concrete is weaker than dry concrete in flexure as well as in compression.

The fact that the writer's results are from specimens of mortar (0-3/8 grading) might conceivably account for his difference in the findings as regards flexure. To the writer this appears improbable. In many other comparisons concretes and mortars have exhibited similar characteristics and have been influenced alike by similar variables. Moreover the "Hatt Tests" were on concrete.

The number of individual specimens tested is relatively small but in the case of the torsional tests two breaks were obtained from each specimen and two or more breaks were obtained from each of the tensile specimens. The work was all very carefully executed and controlled and the individual results were in perfect accord with the group showing in practically every case. The findings from the 2 by 4 in., 3 by 6 in. and 6 by 12 in. compressive specimens were in very close agreement with one another.

The condition at test has a very pronounced effect upon the modulus of elasticity and upon ultimate deformation as well as upon strength. Accurate strain measurements were obtained for all the tests listed and it was found that specimens tested wet (of whatever previous curing) have a higher modulus of elasticity but a much lower ultimate deformation than the corresponding specimens if dry at test.[2] Professors Davis and Troxell[3] have observed the decrease in the modulus of elasticity with air-drying for compressive specimens. The curing and condition at test appear to have little effect upon Poissons ratio.

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[1]Trans. Am. Soc. C. E., Vol. 89 (1926), p. 271.
[2]*Proc.* A. S T. M., Vol. 29 (1929), p. 705.
[3]Same, p. 700.

BY H. F. GONNERMAN (AUTHOR'S CLOSURE)

PROFESSOR GILKEY has pointed out the difficulties which have been encountered in interpreting the results of many previous curing investigations due to failure to test the specimens under uniform moisture conditions. This point cannot be too strongly emphasized as the necessity for having the specimens at the same moisture content when tested, particularly in curing studies, has not generally been appreciated by investigators.

From his own investigations Professor Gilkey has found that beams, compressive and torsional cylinders consistently produce lower strengths when tested in a saturated condition than in a dry or "as cured" condition. This is, no doubt, the case when specimens are wet throughout their entire cross section when tested. In the 1-year tests reported in the writer's paper the strengths of the soaked specimens were always lower than those tested "as cured" or dry, when the length of soaking was sufficient to thoroughly saturate the beams.

The beams tested by Professor Gilkey were much smaller in cross section than the writer's and therefore were more readily saturated in the 24- to 48-hr. soaking period. The mixtures and methods of curing used also tended to shorten the necessary soaking period. Our laboratory is making further studies of methods of soaking beam specimens in order to insure complete saturation at time of test.

Mr. Remler in his discussion of the paper states that:

> A laboratory test of this nature, regardless of how comprehensive, is only a poor criterion by which to judge the practicability of a curing method for concrete roads. The writer knows of no extensive laboratory test on curing which has taken into account the drying effect of the wind and the radiated heat of the sun.

One should be able to evaluate with considerable precision the relative merits of different methods of curing concrete by means of laboratory tests. Results of field and laboratory tests may differ in degree, but the order of merit, at least so far as strength, absorption and resistance to wear are concerned, may be expected to remain the same if the tests are conducted on a comparable basis. The writer has used silicate of soda in four different investigations some of which were carried out in the field where exposure to sun

and wind were involved* and where there was every opportunity for the action described by Mr. Remler to take place. In all of these investigations the results obtained in the field confirm those obtained in the laboratory in showing that this method of curing was little, if any, better than no curing.

Mr. Remler makes the statement that the average laboratory tests have given results with silicate of soda far below actual field tests and refers to the report of the Special Committee on Curing. The writer has studied the data upon which the report referred to was based and found it extremely difficult, as Professor Gilkey pointed out, to evaluate the results in many cases, due to the lack of control of factors other than methods of curing and to the fact that the test programs did not include specimens which were given no curing at all.

Mr. Remler mentions the beneficial effect of curing in the molds but evidently failed to note that the tests in which the specimens were left in the molds were incidental to the main series upon which the conclusions were based.

The fact that the results of wear tests reported with silicate curing do not confirm those obtained in field tests carried out by the U. S. Bureau of Public Roads or in Mr. Remler's laboratory does not necessarily invalidate them. It is significant that the data in Table 4 of the report in the February, 1930 issue of *Public Roads* referred to by Mr. Remler show less wear for concrete given no curing than for that cured with sodium silicate.

In commenting on the absorption tests, Mr. Remler apparently overlooked the significant fact that kerosene was not used to remove the coating from the specimens treated with sodium silicate which showed as high an absorption as uncoated air-cured specimens.

*"Studies of Curing Concrete in a Semi-Arid Climate," by H. F. Gonnerman and C. L. McKesson; Bulletin 15 of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

Discussion of Report of Committee 604

"Winter Concreting Methods"*

Convention Discussion

R. B. Young (Toronto, Ont.)—I was hoping that I would get the same kind of surprise that Mr. Lindau got this afternoon, and have Mr. Johnson come up and take the burden of presenting this report. We had a very animated discussion of winter concreting methods following Mr. Burks' paper. Probably at this time and in this climate, it is not a matter of considerable interest, but there are temperatures of twenty, thirty, forty and fifty below zero in which you have to place concrete. Mr. Johnson has undertaken a preliminary report. It does not attempt to cover the entire field of winter concreting methods, but deals almost entirely with winter concreting methods as applied to building construction. (Following the presentation of the report contributed the following discussion.)

I often hear in our territory that frozen concrete thawed out promptly, not allowed to freeze again is all right. I do not believe it, and I have a reason for not believing it. We have found in the accidental freezing of test specimens, that it is practically impossible to recover more than 80 per cent of the strength of specimens thawed immediately and cured carefully thereafter. In breaking such specimens we always find that moisture forming ice in the concrete leaves what we call turkey tracks; it leaves cavities which, when the concrete again thaws out, are never filled by any future consolidation of the concrete; such voids in the concrete affect the concrete not only as to strength, but what is more important, its resistance to weathering. I have been travelling up and down the country looking at concrete and seeing a lot that was frozen (some said to have been frozen that was not,

*By Robert C. Johnson, author-chairman; Journal, A. C. I., Feb. 1930; Vol. 26, p. 397. Presented at the 26th Annual Convention in Mr. Johnson's absence by R. B. Young, member committee 604.

but did not get proper curing) and I can say that any concrete I have seen that was frozen, was permanently damaged. Though it stood up for a few years, it showed the effects of it in the course of time. While you might recover a part of the strength, you cannot recover all of the properties of that concrete after it is frozen.

Mr. Johnson brings out another point that had not occurred to me, because I have had nothing to do with the type of structure.

One of the great dangers of winter concreting is in the erection of tall reinforced concrete buildings designed for light live loads. In buildings of this class the live load for which the floors are designed are usually less than the construction loads in the erection of the building.

That is an important point. I certainly have seen concrete abused at a time when it was not able to stand it. The concrete was not in position at the time to stand the rough usage it got, and one thing that results from winter concreting occasionally is lack of curing, which puts the concrete in such condition that, when it is first severely exposed to the weather, it is not able to resist that exposure. For example, consider a small mass of concrete—small piers or thin slabs, placed under good conditions. The material is heated and put in at a temperature of 75° to 80° but does not have any great amount of protection after that time. It may be cured for 24 hrs. and not protected from freezing; it hardens and looks like good concrete. That concrete would go through the winter at low temperatures or temperatures below freezing and get no further chance to cure. Spring comes and it is saturated with moisture during the thaws, gets frozen during the night or is acted upon by frost, and it is surprising the number of times that happens, and that concrete goes all to pieces the first spring. I think I examine three examples of that every year, sometimes more. Cold weather concrete has to be cured until it has acquired resistance to weather, as well as strength. I might cite an interesting example; we had a rather extensive slab that showed failure one spring. The engineer in charge of placing mass concrete got a little careless with these thin slabs and did not give them the amount of protection they should have; the result was that the steaming was discontinued in about 12 hrs. In the spring the concrete became saturated, and in a few cold nights and warm days, it made a mess of things. To demonstrate the danger of insufficient curing, we took concrete

blocks, made them up to identical proportions, cured one group for sixteen hours by steam, the same as we had done, cured the others for seventy-two hours as the specifications require and sent them out to a cold storage place and had them frozen and thawed, and after fifteen alternations, those that were cured seventy-two hours showed no effect; those cured sixteen hours by the steam were going to pieces. Mr. Johnson outlines the methods for frost protection for buildings, and in general they are applicable to other types of winter concrete.

I think Mr. Johnson is willing to go a little higher than the standard specifications call for; the temperature limits, I believe, are 50° to 120°. There are difficulties in very cold weather, in handling concrete at 140°; you cannot see because of the fog.

Mr. Johnson has prepared an admirable preliminary report. I know he intends to amplify this or supplement it in a later report.

Louis J. Street (Winnipeg, Man.)—Mr. Johnson would permit a temperature of 120°. My experience has been that it is not safe to go 100°; that some cements, normal at 70° make bad concrete at 110° and above. We were running at about 110° and all of a sudden we noticed one of the loads of concrete had hardened. We shut down the work and began to test the cement by individual bags—30 to 40 of them with some of the cement we could use mixing water at 150° to 160° without flash setting; others would flash set at 100° to 105°. We decided to hold the temperature of the concrete down below one hundred and we had no more trouble. I think it important in winter concreting to find the permissible high temperature at which concrete can be placed. Another point is with the test cylinders. We have had architects and engineers specify that test cylinders be left exposed under job conditions. Invariably those cylinders will test 900 lbs. per sq. in. in seven days. Yet from such cylinders allowed to stand by a radiator and thaw out, we get 50 to 60 lbs. with the same concrete. Winter concrete to be tested, should not be cured under job conditions.

Raymond E. Davis (Berkeley, Cal.)—I have been considerably interested in the remarks of the last speaker with regard to the flash setting of portland cement at such low temperatures. I judge this was a portland cement?

Mr. Street—Yes, portland cement.

Mr. Davis—In testing a very considerable number of cements from various parts of the country, I have never seen cement behave in this manner until well above 150° and then the effect on the strength was only slight until above 180°. I think we must recognize that there is a critical temperature for portland cement, just as there is for high alumina cement, and let us say when we go above that temperature, we will get retrogression in strength, but if in any of our portland cements that critical temperature is in the vicinity of 100° we would certainly have a great deal of trouble in any concrete, particularly in mass work, in dams and structures of that kind, because our temperatures go considerably above 100°. In steam curing we would run into the same conditions. That is a thing of considerable importance and I wanted to bring it out in connection with the curves shown in this report indicating the strength we might expect at different temperatures. I happen to know the conditions under which these tests were made, and I think the results shown ought not to be regarded too seriously, because at the time they were made, several years ago, two things were not appreciated; one was the moisture content of the specimen at the time of the test, and the other, the effect of temperature on the specimen at the time of the test. In a considerable number of tests at the University of California, of concrete in cubic yard masses, we have had temperatures as high as 150° due simply to the heat of the chemical reaction and the early ages have shown no retrogression in strength. We have, for example, with rich cubic yard batches and with insulation on all sides, seen the temperature go 150° from a normal temperature of 70°, giving a differential of approximately 80°. That concrete, instead of being thousand pound concrete in 24 hrs., as it would be had it been hardened in the open air was 3000 lb. concrete in 24 hrs. That was with one particular cement, and our observations in that particular indicate a considerable difference in cements. It might not happen for another cement. Tests were made on two of our California cements; one is very fine-ground—what we nominally call one of our early strength portlands; another is coarse-ground portland cement. With a rich mix in a small mass which went to 150°, the strength of the concrete with one cement was increased in 24 hrs. by reason of this ad-

ditional heat, about 400 lbs. greater than under normal curing conditions, while the other cement gave an increase of 800 lbs. I could cite other examples indicating that there is a considerable difference in this respect between different cements. Before we make definite prophecies as to what might be expected, we ought to make a considerable study of this whole question as to what happens in concrete when it goes to these high temperatures. That introduces the question of steam curing and what happens in concrete in masses when the temperature goes rather high. I have some observations in connection with the construction of a tunnel for the Hetch-Hetchy Water supply in San Francisco, where the temperatures have gone to 180° with portland cement concrete, and we want to know what happens to our concrete under those conditions.

D. D. McGuire (Nashville, Tenn.)—I submit that in Tennessee in an ordinary construction season, the temperature is often well above 125°F. and we do not get flash sets. In 1929 we thought we were running into trouble with a temperature of 120°F. We investigated by making briquettes and cylinders with cement taken directly from the mill at each fifty degrees, descending, and we found but two stages at which we had a set in less than 45 min.

R. S. Hale (Montgomery, Ala.)—Mr. Johnson advocates keeping concrete damp seven days. That sounds fine. In Alabama if we have any contractor who will keep our concrete damp seven days, we don't need any more cure. We have trouble in keeping it damp 24 hrs., and I hope that Mr. Johnson comes to our state and bids on some of our work.

Mr. Young—I think it is not so important what the upper temperature is; I think there are other reasons why we should not go to 120°. There is no particular advantage in putting in concrete at 120° or higher in a big mass, because if you get it in at a moderate temperature it heats itself up and gets too hot anyway. Your temperatures should be fairly moderate, but if you go to thin sections, you do not get that same heating effect; neither do you get any particular benefit from putting it in at 120°. All the expense of raising the temperature from 90° to 120° is money sunk in a hole; better spend it for protection, so I think the dead line might easily be made a hundred.

BY N. L. DOE* (BY LETTER)

UNDER ordinary winter weather conditions it is quite possible to protect concrete walls from freezing by using cork insulation on the outside of the wall forms. On reinforced concrete buildings this shows a considerable advantage if the cork is permanently attached to the wall form as illustrated in the sketch. This method does not require any further expenditure or labor, regarding winter weather until all of the walls are completed.

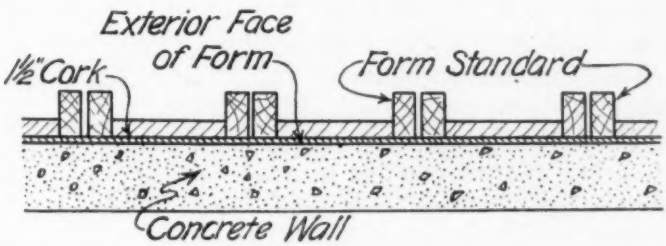

FIG. 1

When cork insulation is applied to the exterior wall forms, in this way, it is assumed that the usual application of heat will be made on the interior of the wall either with salamanders or steam. In the New York territory it was found that with 1½-in. cork, applied to the forms as shown, a difference in temperature of from 25° to 40° would be experienced between the outside atmosphere and the concrete itself. The cost of using cork in this way as an insulating material, assuming that one form was used about eight times, was about two cents per sq. ft. of wall area.

Readers are referred to the JOURNAL *for November 1930 for discussion which may develop. Such discussion should reach the Secretary by September 1, to be considered by Committee in a further report.*

*Member Committee 604.

Discussion of Report of Committee 503

"Proposed Specifications for Supplying, Fabricating and Setting Reinforcing Steel and a 'Steel Setters' Primer'"*

Convention Discussion

R. W. Johnson† (Chicago)—In the absence of Mr. Zabriskie, author-chairman of the committee, I have been asked to present in outline the report of Committee 503, which is a companion committee to 502, just reported by Mr. Lord. For 25 years this Institute has been very active in collecting and disseminating information on all phases of concrete, including construction. The Institute has been particularly active in the design of concrete mixtures and in developing standards for structural designs of reinforced concrete. However, as Mr. Lord so forcefully brought out, this activity has been mostly in the laboratory and in the meetings. We have paid less attention to the construction features than to any other portion of the work. Mr. Lord's specification covers construction details for concrete placing. It was the work of Mr. Zabriskie in preparing his report to cover the construction details in placing reinforcing steel. A considerable portion of concrete placed is reinforced. I am sure the placing of reinforcing steel, its quality, its fabrication, the setting, protection and other incidentals, have been less attended to than any other parts of the work of this society. When we realize there is probably a greater chance for error in placing reinforcing steel than in any other part of construction work, we can be sure we have been neglecting a very important subject. The utmost care is observed in design. The American Concrete Institute has Committee 501 preparing a building code, in minute detail as to the standards for designs, and then when the concrete is actually

*Journal, A. C. I., Feb. 1930; *Proceedings*, Vol. 26, p. 444; formally presented 26th Annual Convention, Feb. 1930.
†Member Committee 503.

placed in the structure, it becomes a matter of guesswork for the steel setter to see that the steel is where it was actually figured in the design. We recall Mr. Lord's paper that accompanied the 1928 report of Committee 501, in which examples were worked out to decimals of inches. Unfortunately in placing reinforcing steel, getting it in, correct to inches, much less than decimals of inches, is very fortunate. This same thought has been brought to my attention by a paper in the January JOURNAL of the AMERICAN CONCRETE INSTITUTE, by Professor Slater, in which he cites tests of four and a half inch slabs. He made measurements as to the placing of the reinforcing steel in those slabs, and found its effect depth varied as much as one hundred per cent. So, Committee 503 attempts to prepare a specification for fabricating and setting reinforcing steel. The proposed specification appears in the February JOURNAL. The rules of the Institute require that to be proposed as a tentative standard, Committee reports must be in print at least 30 days before the annual meeting. Since this was not complied with in the report of Committee 503, it is presented to you at this time for discussion only. Subsequently the Committee will present the report, amended or as first offered, for adoption as a tentative standard. Critic members of the committee are Messrs. Bertin, Johnson, Lord, McKenzie and Thomson. Mr. Zabriskie prepared the original report, which is concurred in, with but a few exceptions noted in the February JOURNAL or in the discussion from Mr. Bertin which I shall present.

*R. L. Bertin** (New York City)—Primarily this specification has to do with the fabricating and setting of reinforcing steel, and for that reason it is my belief that any reference regarding points of design has no place therein.

I am of the opinion that in writing specifications for reinforced concrete too much effort is devoted to standardizing that which cannot and ought not to be standardized. The points I have in mind are the references made to imbedment of straight bars into the supports, lengths of truss bars beyond the supports, location of bends of truss bars, concrete covering of reinforcement, in fact any rule which in any way has to do with the design of the structure.

*Member Committee 503.

If this specification is to apply to design as well, it should be so stated in the title, and special rules written to cover that subject fully, and not for a few isolated cases.

The design of reinforced concrete structures is undergoing a radical change. New methods are appearing in the current literature almost daily for the design of reinforced concrete construction, taking full account of the fact that this type of construction is essentially of a continuous nature. The use of empirical moment coefficients, points of inflections, and what not for beams of approximately equal spans, and carrying a uniform load, are giving way to more consistent methods of design in conformity with Section 710 of the A. C. I. Code.

In general, I feel that any new specification promulgated by the American Concrete Institute should not be at variance with the rules already published in its records.

In connection with the reference contained in the new specification to the effect that certain empirical rules only apply in the absence of definite information from the architect or engineer, this reference impresses me as a compromise between the right and wrong way of detailing. There is no doubt that if these simple rules are contained in these specifications, they will be adopted by many designers to the detriment of the structure, and may eventually reflect adversely and unjustly on the merits of reinforced concrete as a structural material.

Mr. Johnson—Mr. Zabriskie, the author-chairman, has answered this criticism in this way: "Since the detailing of reinforcing bars is a function which must be performed somewhere between the preparation of the design and the actual fabrication of the bars and is related to both, I believe that it is a somewhat debatable point as to where these rules for detailing should be covered. However, inasmuch as the great majority of the detailing work is done by the steel fabricating companies, it appeared to the writer that it was perfectly reasonable to include such rules in this specification; and this view appears to be supported by the other critic members of the committee. I would also refer to the specific assignment of the committee, describing the scope of the proposed specification in which the first item included is recommended practice for bar details. I am therefore

assuming that in creating this committee, it was the wish of the Board of Direction of the Institute to have this subject included. This is the answer to the comments, and while it does not close the subject for consideration, I believe it expresses the viewpoint of the committee and the fact that certain points of design were touched on slightly in the committee report."

In addition to writing the original report for the committee, Mr. Zabriskie prepared a "Steel Setters' Primer," which appears on page 455 of the February JOURNAL. This is written to explain in simple terms the places where reinforcing steel should be used and how it should be placed on the construction project; how the steel should be placed in terms of the structures and in terms of wooden beams, etc., which are readily understood by the ordinary steel setter. It is the thought of the committee that in preparing this concrete steel setters' primer, Mr. Zabriskie has added a document of importance in the records of the American Concrete Institute. It may be used on the construction projects, and it is our hope that contractors, field engineers and inspectors, will find a use for this primer in preparing the way for the better placing of reinforcing steel. Since this is the first time this subject has been before the American Concrete Institute in such detail, we request that we receive any suggestions and comments for altering the specification before it is proposed for adoption next year as a tentative standard. The committee, as a whole, feels deeply indebted to Mr. Zabriskie for a considerable amount of time and effort that he has spent in developing the original specification. I personally want to express my appreciation for the help Mr. Lord and Mr. Thomson gave in the preparation of the report.

W. A. Slater (Bethlehem, Pa.)—Mr. Johnson mentioned the report of the tests in which the steel in the slab was found to be one hundred per cent out of position. The slab was only fifty per cent as thick as it ought to be from the compression surface to the tension reinforcement. I think if a good many slabs were tested and the position of the reinforcing was noted, the designers and constructors would be considerably surprised to find out where the steel really is. My evidence of that in this case is that recently, in talking over this particular test with one of the designers and constructors of the floor, he said that one of the principal modifications in their practice since then had been to

use surer methods of determining the position of the reinforcement in the slab. There is another point that has been brought out in these specifications that I think cannot be emphasized too much, and that is the making of drawings so that they can be interpreted. That was brought forcefully to my attention in the case of the construction job in which, near completion, they found that they had a lot of bars left with no place for them. An investigation showed that the beams required three bars and the designation was such that the steel plates were put in with two; there were only two out of the three bars in the floors throughout the building. The floors showed up all right, but it was not because the steel was there.

A. E. Lindau (Chicago)—Since we are talking about horrible examples and one hundred per cent out of position, I should like to say something about a job I heard of a good many years ago in which in a flat slab construction of a reservoir roof, the bent bars were sent to the job intended as a tensile reinforcement, principally, in the dome where there was tension towards the top. They were put in the job in a reversed position. I do not know how many per cent out of line that would be, it must be over a hundred per cent, I should say, because it was in the wrong place entirely.

Harvey Ace (Louisville, Ky.)—Experience as a designer and field inspector of various types of buildings, leads me to believe that a man is not a field inspector or a field engineer who has not been a designer. A man without designing experience always has bars that don't fit on the job. I recommend that there be some consideration given to recommending that field engineers be required to have had three, four or five years' experience as designers.

Professor Slater—I think a specification for the disposal of left over bars would be a good thing.

E. O. Sweetser (St. Louis, Mo.)—It seems to me that this remark about trained inspectors should be given attention. I understand that in certain countries of Europe, it is customary to train inspectors for reinforced concrete work; that they go through a long period of intense training and careful study, and finally pass an examination. They are then certified inspectors

and salaries are paid which make it a desirable position to hold. This comment comes to me from an engineer and designer who was protesting very strenuously against the great amount of care given to specification writing and the great amount of care given to design, and all this thrown to the winds by inadequate inspection.

F. R. McMillan (Chicago)—While we are speaking about the qualifications for field inspector, a gentleman who sits near me here has had about 30 years' experience as a building inspector, and he says that he would suggest that designing engineers be given a little field training also.

Louis Clousing (Minneapolis, Minn.)—In our city we find that where the engineer who designed the work gets on the job, it makes quite a difference in the job. I think an engineer ought to see his own job, and a good many times the work is done without the designing engineer ever getting near the job.

Readers are referred to the JOURNAL *for November 1930, for discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930 to be considered by the Committee in a further report.*

Discussion of Report of Committee 504

"Proposed Specifications for Ready-Mixed Concrete"*

Convention Discussion

Miles N. Clair†, (Boston)—I do not know whether the Program committee realized the connection that Mr. Burks' paper‡ has with what I have to say, but I was privileged to see his work at Chute a Caron, and I think that his operations up there have a good deal to do with the subject of the work of Committee 504. Mr. Burks has a wonderful central mixing plant and one of the men here brought out that fact by asking him something about the transportation of concrete. His plant puts out, I imagine, something over a thousand yards a day, delivering it to a point, as he stated, about four miles away, and his problems, in a way, are the problems of a central mixing plant.

The report, not of Committee 504 but of the author-chairman, is published in the February issue of the Journal of the Institute.

The committee has been in the process of organization since July 1929; at present is composed of 10 members about evenly divided between plant operators and practicing engineers. No meetings have been held and all the discussion has been by mail.

It is believed that the original designation of the committee as having to do with centrally mixed concrete inadequately describes its activities and the title should be changed to "Specifications for Ready-Mixed Concrete." I respectfully ask that proper action be taken to bring about this change of designation. The committee will endeavor to prepare a definition of ready-mixed concrete.

The author-chairman was asked to consider the problem of centrally mixed concrete from the standpoint of the user and to

*Journal A. C. I., February, 1930 (*Proceedings*, Vol. 26, p. 467).

†Chairman Committee 504, in formal presentation of report to 26th Annual Convention, Feb. 1930.

‡"Concreting Methods at Chute a Caron Dam," by I. E. Burks, Journal A. C. I., Feb. 1930 presented earlier at same convention.

draw up either specifications for such concrete or fundamental considerations leading to specifications for such concrete. A little study made it apparent that for many cases the usual specification for concrete provided ample protection for the user. The usual specification for concrete however does not permit the utilization of all of the advantages of the ready-mixed concrete operation, particularly with regard to the time elapsing between mixing and placing and the control of the quality of the concrete.

A specification outline was prepared and presented to the critic members of the committee and as a result of the constructive suggestions received a new outline was prepared which is presented with this report. The reactions of the committee to this revised specification have not been obtained and it is presented to the society by the author-chairman at this time, so as to benefit by open discussion on the floor of the convention. It is particularly hoped that such members of the committee as are present will present their views.

The specification outline covers under the term "ready-mixed concrete" both the central plant-mixed and truck-mixed systems. Materials are required to meet the usual specifications as to quality and changes are made subject to the approval of the architect and engineer and facilities are required at the plant for making tests of materials readily.

Concrete quality is specified on the basis of compressive strength as determined by tests on the particular materials. The only limitations on the method of proportioning to obtain this strength are that it will give concrete uniformly of the quality desired and that adequate test data are available to enable the engineer to determine whether the quantities used are sufficient for the purpose. A check on the quality as made is obtained by requiring compression tests of the concrete. This requirement may not be strictly necessary in all cases for concrete from a well established plant and with the usual classes of concrete.

The central mixing system and the truck mixing system are considered separately in the section on mixing and transporting. Central plant-mixed concrete is required to be mixed properly before being loaded into the transporting vehicle as the mixing action of the agitating apparatus is considered nil. Vehicles to

transport wet concrete are required to be tight so that water or other ingredients cannot be lost. Ordinary non-agitating trucks are permitted when the consistency is such that segregation will not occur. A consistency, indicated by a 2½-in. maximum slump is suggested as proper. The time between the addition of the water to the cement and the placing of the concrete has been limited to one hour. This is suggested as a reasonable limit although specifications are in use which have limits from 30 minutes to no limit except that the concrete shall be workable when placed.

Additional requirements are given for truck-mixed concrete so as to assure the engineer or architect of good concrete. Certain types of truck mixers have difficulty in mixing properly batches in which the materials are not uniformly admitted to the drum. Attention is drawn to the importance of this clause in the specification. The same limitation is set up as to time elapsing between admission of water to the cement and the time of placing as for plant-mixed concrete. Unless the aggregates are entirely dry this means the 1-hr. period starts with the loading of the cement and aggregates into the truck drum.

The problems of proper serving of the job and of placing have been given some consideration in the section headed "Delivery," where limitations are placed on the plant capacity and the interval between batches.

The author-chairman intends to study the items concerning which there is at present much difference of opinion and to endeavor with the assistance of the committee and others to obtain information which will permit placing proper limitations on those doubtful factors.

I suggest that action on this proposed specification be delayed until the next annual meeting so that suggestions and criticisms of the other members of the committee and of the membership at large may be given proper weight in the preparation of the specification. We feel the subject needs a good deal of discussion before any action is taken towards its adoption.

Chairman—The suggestion by Mr. Clair that the report be freely discussed and then passed on until next year for further consideration, seems to be a good one.

*Russell S. Greenman** (Albany)—In the "Proposed Specifications for Ready-Mixed Concrete" an excellent base has been laid for consideration of essential requirements. Some points offer opportunity for discussion.

This relatively new product, commercially prepared ready-mixed concrete, has been so built upon one quality alone—namely, the production of a concrete of merit—that in writing specifications for it, the thought must be kept in mind that these shall be very largely just formalities of expression. This thought appears in various forms in other discussions which have been presented for our consideration.

If this is so, then such requirements as section 3-d (first sentence) and section 3-e of the proposed specifications are asking for too frequent, formal, certified statements.

These proposed specifications further presuppose that no company enters the business of producing ready-mixed concrete without provision for a well-equipped plant, capable of producing uniform measurements of quantities of materials; also that it shall use all available means for proper design and control of the various mixes and types of concrete.

Anything else but this would be business suicide. To suggest that there is need for a requirement that each load of ready-mixed concrete shall be accompanied by a certified statement tends either to demand an unnecessary formality or to make such a statement become a farcical one. The suggestion that one sample be taken for each fifty cubic yards delivered becomes both burdensome and unnecessary. Do not misunderstand me and think that I do not believe that the architect or engineer should not be given the data which will show the quantities of materials used to produce certain qualities or that tests should not be made in order to check these factors. Some such data must be made available and should be given when desired. But the frequency and number of certifications and tests can hardly be as often and as large in numbers as suggested.

For example—Besides other deliveries one company delivered to one contract 273 cu. yds. in one day and to another contract 256 cu. yds. in another day. Now in either day, with trucks

*Mr. Greenman's discussion, as a member of the committee, was presented in the author's absence, by Mr. Clair.

handling three yards each, the first job would have required 91 certificates and six sets of samples; and the other would have required 85 certificates and five sets of samples. How could any responsible producing company expect to win the steady approval of users of their product if it did not agree—upon acceptance of an order and without repeated certificates, that such type of concrete as ordered would be furnished regularly and uniformly? By early tests of a type of mix, its quality can be determined and then occasional check tests can be made but to demand that these tests shall be on every 50 cu. yds. is accomplishing no real good results.

To eliminate or at least to modify the first sentence of section 3-d, and to modify section 3-e so as to make the number of tests flexible, would seem to be advisable.

This brief discussion can be summed up largely by stating the thought that no wise operators of a commercial ready-mixed concrete company can permit any operation or procedure which is not based on most conscientious and systematic attention to detail of quality and uniformity—and checked by tests; therefore acceptance of an order should be equivalent to furnishing a certified statement for each truck load delivered.

Mr. Clair—Possibly the word "certification" here is a poor one; what is meant is a ticket such as anyone would send along with a load of concrete, as a record of its receipt, and just what is in the load. The report does not imply a certificate before a notary public. Mr. Greenman has taken the requirements to mean a set of specimens on every fifty yards. The requirement is a specimen on fifty yards of concrete.

*H. F. Thomson** (St. Louis, Mo.)—My point of view is that of the operator with some engineering background. I would call attention to one particular feature that, it seems to me, the Institute should pay some attention to, not necessarily confined to the work of this committee. We all recognize that the introduction of ready-mixed concrete offers an improvement—an opportunity for a degree of control hard to obtain in ordinary job-mixing, through the accuracy of proportioning, the concentration of purchasing, which means a certain influence in connection with the grading of the aggregate and the water

*Member Committee 504.

control. Combined, those items all mean that the centrally mixed concrete (and by centrally mixed concrete I mean the truck-mixed as well) when it is under proper control, will give a much higher strength than is ordinarily anticipated in job mixing. If a structure is designed for 2000-lb. concrete and that concrete is ordered as, say, one-two-four concrete, the central plant if it delivers concrete with that proportion—with that cement content—will actually be furnishing very much higher strength, something approaching 3000 lbs., most frequently. We are approaching a point where the degree of control offered by the ready-mixed operation suggests a possible economy either in using less concrete or permitting the ready-mixed operator to furnish concrete of a given strength as actually contemplated in the design. I merely take occasion to call attention to the opportunity, some day, of setting up a specification, a so-called strength specification, which will be based on some measure of workability rather than on the water-cement ratio. Then the ready-mix operator will be able to utilize the efficiency of his plant and realize the economy of using merely the quantity of the cement which is necessary to furnish the strength at the workability specified.

That is a little outside the scope of work of this committee, but it is a matter which should be kept in mind, as the increased strength for a given quantity of cement is being attained by the ready-mixed operator everywhere. If the operator is not to be handicapped in competition with the proportion mixes of one-two-four, etc., there should be some recognition of the possibility of operators furnishing the strength desired at a given workability.

*W. E. Hart** (Chicago)—The chairman of this committee has asked the further consideration of the specification by the Institute in the coming year. I think it is very important for a great many reasons. The Institute has set up definite standards for making concrete; the possibilities in centrally-mixed concrete are limited in two directions; you may either destroy the good that has been done in past years, or you may make a better concrete than is being made at the present time. I think it is essential that we consider very carefully and watch in practice the specifi-

*Member Committee 504.

cation suggested by Mr. Clair. There are certain changes that, of necessity, have to be made, as mentioned by Mr. Clair. I have a few that I wish to offer later, to supplement this discussion when published.

It is essential that we safeguard this new industry; it is springing up in every part of the country. Two years ago there were about ten or twelve central mixing plants; last year there were more than a hundred. I cannot say how many there are this year; there must be at least 150. (Mr. Hart later proposed the following changes.)

It is suggested that paragraphs (page 469-70 of the February JOURNAL) be changed to read as follows:

[3 (c)] The method of measurement of concrete materials, excepting cement, shall be by weight, so that the proportions can be adjusted readily to assure concrete of the quality specified. The operation of the plant, etc.

[4 (a) 1] The concrete shall be properly mixed before being loaded into the transporting vehicle. A minimum mixing time of one minute shall be required when the concrete is transported in truck bodies in which provision is made for additional mixing or agitation. When other types of bodies are used, the minimum mixing time shall be increased at least 50 per cent.

[4 (a) 3] Non-agitating trucks shall not be used to transport centrally mixed concrete when the consistency of the concrete is wetter than a 2½-in. slump.

[4 (a) 4] Centrally mixed concrete shall not be transported in any case for a period in excess of three hours.

[4 (b) 1] Add the following note:

"Note: It is suggested that the cement and aggregates be fed simultaneously into the end loading type of truck mixer. For the top loading type of mixer this method of charging is not so essential, but is preferred and recommended."

[4 (b) 2] Trucks used for transporting and mixing concrete shall be so powered as to insure proper mixing while in transit at the manufacturer's recommended speed.

[4 (b) 3] It is suggested that the time period be changed from one hour to three hours. Also add the following: "In case of failure of mixer power, the batch shall be discharged from the truck within one hour."

[4 (b) 4] Means shall be provided so that the quantity of water added to the mix can be accurately measured and checked.

Add the following paragraphs:

[4 (b) 5—Mixing] It is recommended that truck mixers be so equipped that the concrete cannot be discharged until a given time has elapsed, or a certain number of revolutions of the drum have been made. Until further data are available, it is recommended that at least 4 minutes mixing time be required, or that the drum shall make at least 36 revolutions, prior to discharge of the batch.

[4 (b) 6—Rated Capacity] Note: With the present knowledge available, it is recommended that the working capacity of any mixer shall not exceed ½ of the total volume of the drum, including the cone, where used.

[4 (b) 7] It is suggested that this paragraph be placed ahead of the present Paragraph 1.

"All concrete mixed in transit shall be mixed in a revolving drum type of mixer."

[5 (b)] It is suggested that three words "for a pour" be eliminated. This paragraph would then read as follows:

"(b) The interval between batches shall not exceed thirty minutes or in any case, etc."

*Morris Brooks**—We have given pre-mixed concrete much serious thought, and after exhaustive tests have arrived at the conclusion that concrete scientifically mixed in a plant conducted by responsible owners, whose continuance in business is dependent on the production of uniformly first-class concrete, delivered in agitating trucks, and placed in the forms within certain required time limits, and under proper temperatures, has the effect of eliminating much of the guess work in the use of concrete.

I have prepared a set of regulations governing the use of pre-mixed concrete in Philadelphia, and would like to present these regulations for your consideration.

REGULATIONS GOVERNING USE OF CONCRETE MADE AT CENTRAL MIXING PLANTS

Definition: Concrete which is mixed at such distances from point of depositing as to necessitate the use of approved vehicles other than buggies, chutes, or similar contrivances, is considered as central-plant-mixed concrete.

As the result of field and laboratory tests, directed by and under the supervision of the Bureau of Building Inspection, the use of concrete made at central mixing plants is approved for construction work in Philadelphia, subject to the following conditions and requirements.

1. Ready-mixed concrete shall be made in plants located within the City of Philadelphia. Representatives of the Bureaus of Building Inspection shall at all times have unrestricted access to the plants, materials, records and all other data pertaining to mixing, transferring, use and inspection of ready-mixed concrete.

2. All materials entering into the mix and the concrete mixed at a central plant shall be certified by thorough and competent independent inspection approved by the Bureau of Building Inspection.

3. The central plant shall be equipped to properly and accurately measure the materials entering into the concrete mix. Owners shall present to the Bureau of Building Inspection evidences of their competency to carry out in full the requirements of this approval.

*Chief, Bureau of Building Inspection, Philadelphia.

4. The cement shall be measured by weight; and when purchased in sacks shall be taken from original 94-pound bags as supplied by the manufacturer. No bag of cement shall be divided for mixing. Aggregates shall be measured by weight. Ingredients shall be measured by weight when in the dry state, and by volume in calibrated containers when in the liquid state. Under no conditions shall materials be measured by "shovel count." The plant shall be equipped with all necessary apparatus for verifying scales and measuring devices. Concrete shall be mixed in an approved rotary mixer equipped with a timing device which will automatically lock the discharge lever during the mixing period and release it at the end of the full time of mixing.

5. The concrete shall be mixed one and one-half minutes as a minimum, with the drum rotating an approved number of revolutions. Larger batches of concrete may be required to have a longer time for mixing.

6. Cement and aggregates shall meet the current A. S. T. M. specifications. Each shipment of cement shall be tested. Complete tests of the aggregates shall be made daily. The water shall be clean, free from oil, vegetable substances, etc., and the tensile strength of 1:3 mortar gauged with this water shall be not less than that developed with distilled water when mixed in the same proportion using Ottawa sand. Tests for moisture content, clay, silt, and organic matter and sieve analyses of the aggregates shall be made at intervals sufficient to control the quality of products and proportions and consistency. Aggregates must be tested for moisture content within three hours. Tests of aggregates for organic matter shall be made from deliveries before they are unloaded in the plant.

7. Every truck load of aggregate or concrete mixed in a central mixing plant to be used in a structure erected under the jurisdiction of the Bureau of Building Inspection shall be certified as having the required and approved proportions, and that it is thoroughly mixed. This certification shall be made on a written report made by an independent inspector engineer stationed at the plant, who has given to the Bureau of Building Inspection satisfactory evidence of his experience and qualifications to make such certification over the individual signature of the inspector certifying. The responsibility for the filing of this information each day central mixed concrete is mixed and transferred rests with the plant, and failure to so certify it automatically rejects the product as acceptable for use in structures.

8. The central mixed concrete shall be discharged into clean, water-tight bodies, and shall be hauled in rotary or agitator trucks approved by this Bureau.

9. Concrete from central mixed plants shall be deposited at the site where it is intended for use at a period not to exceed 60 minutes from the time it leaves the central plant mixer and deposited in forms within 30 minutes. The time of initial set of the cement used in the mix shall always be greater by certified test than the elapsed time between mixing of the concrete and the depositing in the forms.

10. A delivery ticket shall accompany each load of ready-mixed concrete, setting forth the class or type of concrete being delivered, and the exact time

the concrete was discharged from the central plant mixer into the delivery truck. This ticket shall be signed by the inspector at the plant. Upon reaching the job each certified load of concrete shall have the exact time of dumping noted on the ticket and a check mark placed opposite the class of concrete delivered. The resident inspector on the job shall certify that the class and quality of concrete delivered conform to the Laws and Regulations of the Bureau of Building Inspection. These tickets shall be filed with the plant, and at all times be accessible for examination by the Bureau of Building Inspection.

11. In freezing weather, or when temperatures below 40° F. prevail, concrete shall be delivered and deposited in the forms at temperatures not less than 50° F., nor more than 100° F.

12. When in the judgment of the Bureau of Building Inspection load tests are required they shall be of such number as is adjudged necessary. They shall be made in accordance with the laws and regulations governing such tests and test procedure in the City of Philadelphia.

13. Wherever in the judgment of the Bureau of Building Inspection it is for the best interests of construction not to use plant-mixed concrete, this approval does not apply.

14. All laboratory tests shall be made by laboratories of recognized standing, and acceptable to the Bureau of Building Inspection.

15. Failure to comply with the provisions of the terms of this approval will be considered as withdrawal of approval without further notice.

V. E. Schevenell (Memphis, Tenn.)—I am also an operator of a ready mixed concrete plant, and my reaction to the proposed specification is this: in making specifications for ready-mixed concrete, I think we should separate the centrally-mixed concrete and the truck-mixed concrete. My observation is that about ten different types of trucks now attempt to mix concrete. Some of them mix it very well, and some of them not so good. For that reason, two specifications should be considered by the Institute. The batch mixer is of course used by all the central mixing plants. If you remember, I think it was in 1850 that the first concrete mixer was made, and in 1875 the continuous mixer was made, and that mixer has now been discarded by all engineers, and in 1900 or a little earlier, the batch mixer appeared. If we permit any and every type of truck mixer to come into the picture, without a specification, the Institute will be making a serious mistake. To make two specifications this committee will require more time, but I think time will be well spent in investigating the different types of mixing trucks. That does not mean the agitating truck. The agitating truck of course receives batch

mixed concrete and simply agitates it, but the so-called mixing trucks do not.

Mr. Brooks—In Philadelphia we have approved ready mixed concrete, but it must be delivered in an agitating truck; that is the only truck we approve there; the concrete must be continually agitated until it goes into the hopper; it must be mixed at a central mixing plant and agitated until it arrives at the job.

R. F. Leftwich (New York City)—Mr. Thomson spoke as a producer of ready mixed concrete, and as a user of ready-mixed concrete in quite a large portion of the United States, we have come in contact with most of the ready-mix plants in the west, and it has been my privilege to go to most of those plants, and at those not prepared to deliver what we wanted, to get them to deliver what we wanted. Mr. Clair certainly has a model system of ready-mixed concrete in Boston, which I have had the privilege of investigating recently, and if we could get the other mixers to follow their example, the contractors would not have much trouble. Our experience is that the average ready-mix plant (which does not apply to members of the Institute) does not know what proportion to put in a mixer to turn out a yard of concrete. In one instance we ordered 10 yards of ready-mixed concrete, and they sent ten truckloads, which turned out to be eight yards. We found that the plant was trying to turn out a yard of concrete with a three-quarter yard mixer. The operator put in the central mixing plant for the people who live in that vicinity because they did not want to set up a mixer every time they needed some concrete for a sidewalk, and he turned out a batch of concrete and called it a yard. That is an extreme case. Another experience was with one of the best central mixing plants I have seen. We had four jobs in a city where we used between two and three thousand cubic yards on each. We used a central mixing plant, for the simple reason that when we wanted yardage, we wanted a very large yardage at widely separated jobs on the same day and we could not afford to put up a plant to turn out this yardage on each of these jobs. Our average loss was twelve and a half percent shrinkage. That is, the yardage delivered to us exceeded the yardage measured in steel caissons by 12 percent. At one job the shrinkage went as high as 17 percent. We went to the

plant operator and discussed it with him, and found out why we were not getting the yardage. He did not agree with certain people in town as to the weight of his gravel and sand. They got together and said that the sand and gravel should weigh 100 lbs., but the sand actually weighed 82 lbs. and the gravel 95 lbs. We took it up with this organization, but they will not guarantee to give us a guaranteed yardage. It happens to be a 1:1:2 mix and the variations did not amount to much, but it did amount to a great deal in dollars and cents when the finished yardage was checked up.

I think one of the greatest services that the committee on this particular specification can render the consumer of concrete, who is as vitally interested as anyone else, is to see that these plants learn how to make a yard of concrete—what proportion of the different materials to put in. We have paid for it, paid dearly, and in Boston they can give you a 1:1:2, 1:1:3, 1:1½:3 or a 1:2:4, or a strength concrete inspected by laboratories. The city checks it and you get your yardage, and if these other plants would do the same way, it would not cost them any more money and they would get more business. Another point Mr. Thomson brought out that amused me was the statement about the ready-mixed concrete giving more strength than the average job plant. I agree with him thoroughly that in a plant mixing two or three yards of concrete at a time, you can get better concrete than in a 4-yd. mixer out on the job operated by less skilled help. The reason we get a stronger concrete is that we get a richer mix by reason of failure to figure shrinkage of materials. I am pretty sure that is not the case with Mr. Thomson's plant, because he has a very good reputation among us. The work that can be done in educating concrete mixing plants to give guaranteed yardage in place is very important.

J. C. Pearson (Allentown, Pa.)—It is my impression that this very excellent concrete that is made in Boston—I would like to ask Mr. Clair whether I am correct—would be acceptable in Philadelphia?

Mr. Clair—Concrete made in Boston would, I think, without question, be acceptable in Philadelphia. I think that it meets the requirements except for one thing, that is, Boston would not have any limitation at present upon the length of haul.

Mr. Pearson—I understand that a large part of the operations in Boston is not with centrally-mixed concrete.

Mr. Clair—That is true. In Boston we have an unusual situation; there is a central mixing plant and a central batching plant or truck mixing system. The regulations made and used may appear lenient in some cases and strict in others. In Boston you can make concrete to a strength specification if you have special permission. That is not yet true in most cities. The code itself says that you shall mix one to six parts, one part of cement to six parts of aggregate, and that shall give you twenty-two hundred pound concrete. However, if you are responsible, you can go to the commissioner and request permission on your particular job, to use a strength specification.

Herbert Coffman (Philadelphia)—The speaker who told about the shortage of twelve or seventeen percent of his yield of delivered concrete shows the importance of having scientific control in a central mixing plant, and what comes of the lack of it in a plant that puts out concrete like that. In the specification, the weight of sand and gravel (the A. S. T. M. specification for the weight of aggregate for concrete) are determined on the dry rodded basis. You should have a permanent basis for your yardage on the volume method and control of the water. There are two or three methods for determining very easily the volume of concrete. You can take the gross weight and net weight of the truck coming in and going out and check it by the number of cubic feet; there are many ways of checking the yardage of the concrete that is being furnished. The only way to do that is to have a scientific basis for the man going into the central mixing business and draw up certain specifications, and he can state whether or not he is operating under the tentative or approved specification of the American Concrete Institute.

A. E. Lindau (Chicago, Ill.)—In connection with the subject of the yield of centrally-mixed concrete, I would like to refer you to Mr. Burks' paper, page 334 of the February JOURNAL of the American Concrete Institute, in which he says: "On January 1, 1930, the total concrete placed, according to batch-count, was 286,625 cu. yds. The engineers' figures at this time showed

286,658 cu. yds., or about seven-tenths of one percent more than the batch count." Apparently Mr. Burks knows how to get the batch count in cubic yards of concrete.

Ernest Ashton (Allentown, Pa.)—The advent of central mixing plants presents an interesting problem to the cement manufacturer, which is somewhat foreign to investigations heretofore conducted. On several occasions we have been called into consultation on centrally-mixed concrete which upon being placed exhibited a phenomenon known in physical chemistry as syneresis which the average layman would call bleeding. Free clear water appears on the surface tending to form small rivulets on the slopes preventing or at least halting the operation of finishing.

We have commenced a rather definite study of this question and hope both the central mixing plant operators and users will bear with us until a solution is arrived at.

Mr. Clair—Since part of the discussion appears to be in regard to the quantities that are delivered, it may be of interest to know what our experience has been in Boston. We have taken the concrete from certain plants and not only computed what we should get out of the batch, but have measured what we got out of the batch. We have done that a number of times, and it is supposed to be done more often. If there is any question in anybody's mind at the time the concrete is being delivered as to whether or not they are getting a yard or two or three yards, they load up a truck of one yard capacity and take it out there and measure the next truck that comes along and compare the two. That does not answer the question as far as the contractor is concerned. Naturally, when he was estimating the job, he computed the volume in those blocks of concrete and figured that he needed so much concrete and thought he would go out and buy it instead of making it himself, and he did buy it, and when he gets all through, he gets a bill for five or ten percent more than what he computed was in that block. Well, I consider in general that his surprise at that is rather vague, because he knows that some concrete is spilled, he knows that the forms give and he knows that the subgrade is not where it is shown on the plan. Our checkup has shown that these three items, plus shrinkage, almost invariably explain where the extra concrete has gone.

Mr. Thomson—There is one point in the specification as reported by Mr. Clair that, it occurs to me, has not been appreciated in its entire significance; that the centrally-mixed concrete hauled in the non-agitating truck is limited to concrete of two and a half inch slump or less. We happen to operate both types of service, that is, the centrally-mixed service without agitation and a truck-mixing or agitator service, and we are frequently asked which we prefer. Actually we do not consider that the two types of operation are antagonistic or competitive. For stiff mixes, as contemplated in this specification, which says 2½-in. slump, maximum, we prefer to use the central mixer without agitation, and we have served jobs at rather unusual distances with entire satisfaction under these conditions. We served one of the largest railroads in the country, a job of about 2500 cu. yds., at a distance of about 14 miles from the plant, without any difficulty at all. But for the wetter mixes—and I assume that is where the Philadelphia specification comes in—the wetter mixes you need in building, of over 2½-in. slump, then agitation or truck-mixing is to be desired, so that the two types of operation contemplated in this specification, do not necessarily present themselves as competitive, but supplementary, and by offering them both a company is in position to cover the entire field.

Readers are referred to the JOURNAL *for November 1930 (Proceedings Vol. 27) for further discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930, if it is to be considered by the Committee in a further report.*

Discussion of Report of Committee 707

"Design of Concrete Products Plants for Single or Multiple Shift Operation"*

Convention Discussion

D. R. Collins (Milwaukee)—Mr. Wilk's conclusions agree to a great extent with observations in Milwaukee and vicinity in the last two years. Ever since the problem of multiple shift operation was discussed on the floor of the Wisconsin Concrete Products Association convention two years ago, there has been much talk and several experiments along this line of operation.

The plants which have tried operating more than one shift have found it unsuccessful for two primary reasons:

First—The harsh nature of the material passing through a concrete block machine. The very texture of this material and the act of compressing it into a concrete block makes for more wear and tear than practically any other material in a manufacturing process. Stones and pebbles sticking at various places throughout the machine quite often throw it out of alignment and the constant rubbing of the concrete wears out the various parts of the mold box very quickly. For this reason it is necessary to watch machines very carefully and repair them frequently if the kind of product is to be made that is now demanded by mason contractors.

Second—Curing operations as now conducted require an outlay of pallets, racks, curing chambers and possibly boilers, very nearly double the amount needed for single shift operation.

What is probably the most efficient concrete products plant in Milwaukee county, expanded its market so quickly that additional facilities were needed.

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*By Benjamin Wilk, Author-Chairman, Committee 707; Journal A. C. I., Feb. 1930, *Proceedings*, Vol. 26, p. 498.

†Critic Member, Committee 707; discussion presented at 26th Annual Convention.

Alfred H. Boelk was, at that time, plant superintendent and was producing concrete block in a single shift as efficiently, and I believe, with as low a production cost as has ever been accomplished in the county.

Taking a few key men from his day shift and selecting a well educated and intelligent working foreman he attempted to build up a night shift to remedy the situation his company has worked itself into. The observations that follow are largely his and are based on his actual experience in operating two shifts a day in a very efficient concrete products plant. He states:

The theoretical view of a plant producing 500,000 units annually and increasing to 1,000,000 annually by multiple shift is, in my estimation, an impossibility unless broad exceptions are made in the second or night shift. It is true that this method of operation would cut overhead costs provided the plant were properly arranged and if this plant *is* arranged for multiple shift and is running full capacity there must be a steady market to take care of the production. Without this steady market overhead will be far greater with a multiple shift plant in operating continuously than a single shift plant arranged to produce units as economically as possible in the regular 10-hr. day supplemented with arrangements to operate 12 or 14 hrs. daily when necessary.

I believe a plant arranged properly for a single shift with provisions made for overtime work is more economical than a multiple shift plant because of the lower initial investment, if this plant, with proper management, can produce enough units by overtime work to take care of even abnormal demands.

It has been my experience that a far better unit is manufactured by single shift than a multiple shift. My experience has been with both. A quality unit is absolutely essential in this day of highly competitive selling and should be a primary requisite in determining whether or not two shifts are to be operated. My reasons for making this statement are:

Many small blemishes and cracks which can be seen by day cannot be seen at night with artificial light. This often leads to such units going into curing rooms and out into the yard where these defects are cause of breakage in handling or trucking. Density of the block, an important factor secured only by proper tamping can be determined more closely at day than at night. This has not only been proven to me by night work, but by winter work, a great deal of which is done by artificial light.

Help is more efficient and reliable on a day shift than at night. The night crew is necessarily a limited one and if one or two men fail to put in appearance it is practically impossible to secure additional help for night work. This not only throws the night shift out of balance but results in a loss instead of a profit for that night's work.

Facilities for repairing and replacing machinery are far greater by day than by night. Unless the concrete products plant has a well equipped machine

shop, minor adjustments or welding which could be secured from a nearby machinist quite readily during day operation cannot be had at all during the night. This results in a delay and loss of probably a great portion of the night's work.

Production does not seem to hold up during the night as it does during the day and at the same time, the lives of the machines are materially shortened.

Racks and pallets must be considered. Unless some high speed curing system is perfected more racks and pallets will have to be purchased.

Good help at night will cost more money per hour than day workers. In addition, these men are not as regular in attendance as day men and are less efficient in their work, running up the unit cost for night production.

Inexperienced help, which is often necessary to be used for night work, is inclined to abuse the machinery to such an extent that the life of it is shortened at least 15 to 25 per cent.

Accidents seem to occur much more easily and with greater frequency during the night shift than during the day.

A machine capable of turning out 2,000 units in 10 hrs. with two men at a labor cost on machine only of $12.00, can turn out 2,800 units in 14 hours at a labor cost of $16.80. My experience has been that a night crew working 10 hrs. will turn out between 1600 to 1800 units in this same length of time, 10 hours. Figuring the day shift at 60c an hour and the night at 70c, the rate it was necessary for me to pay, the average cost per unit for the day's shift was .0128 while the night shift cost .0167. I found that there seems to be a period during night operation, coming shortly after midnight, when the morale of the crew apparently is disorganized and during which time production slows up very greatly. I have found that by working my crew a full 10 hrs. and following the practice of having the mixer man on the job one-half hour early every morning so that when the seven o'clock whistle blows operations may be started at once, production speeds up considerably.

Another practice that I found that makes considerable difference is that of running machines up to the closing whistle at night and have a man then take charge of them, scrape off the wet concrete and follow through on each machine, carefully checking it over, oiling and greasing it, making any minor repairs and getting ready for the next day's operation.

The experience outlined by Mr. Boelk makes, I believe, a very conclusive case in favor of the report by Mr. Wilk.

W. D. M. Allan (Chicago)—Mr. Wilk, author-chairman of Committee 707, has introduced a problem, the solution of which strikes deep at the fundamentals of efficient plant operation and, in turn, at the economic success of the concrete products industry. The idea of multiple shift operation has aroused as much thought and discussion among leading products men as was started seven years ago by the subject of efficient aggregate grading. From

Mr. Wilk's paper, there will doubtless continue for several months, if not years, a widespread discussion regarding the relative efficiency of operating a small plant 16, 18 or 20 hours a day in two shifts, or operating a large one, 8 9 or 10 hours a day on one shift.

The question of multiple shift operation is, in my opinion, only one phase of the larger question of efficient production. The concrete products industry has grown so rapidly that very little attention has been given to the true fundamentals of mechanical efficiency in the plants. The discussion of the number of hours or shifts per day a plant should be operated cannot help but lead to a study of piece work and all of its complications, the size of machines, efficiency of molding devices, methods of curing and numerous other subjects.

If the Institute should undertake such an investigation, the dispatch with which its work is done and the value of its findings will depend on selecting men with special training for the committee. A study of efficient plant operation can be divided into several phases, each of which can be studied by sub-committees.

At this time, any discussion of multiple shift operation must be partially theoretical because very few manufacturers have had first hand experience over a long time to make a valuable decision on this method of production. In discussions I have had with manufacturers who are favorable or opposed, the statement is frequently made, "Oh, we've tried out the plan and it's the only thing," or "It's no good and won't work." Further questioning almost always brings out the fact that multiple shift was tried for a few weeks, or even months, but in no case long enough to justify any well founded conclusion.

It is quite possible that the concrete products industry after exhaustive study will arrive at Mr. Wilk's conclusion regarding multiple shift operation. "On the basis of this comparison, it is evident that in the concrete products industry the single shift plant is to be preferred over the multiple shift plant." However, there is not yet sufficient data, experience or study to warrant taking this conclusion as final and even though the conclusion might apply in the type of plant he has selected, it is quite possible that a plant making a smaller range of products may find

multiple shift operation very much to its advantage. I believe that Mr. Wilk would be disappointed if this most important subject were disposed of without thorough investigation.

There would never have been any thought or talk about multiple shift operation were it not that many leading manufacturers realize that the concrete products industry is not making the progress that it should. This is in spite of the 700 per cent increase in the use of the product in the last ten years. Each year since the war until 1929, the output has increased over the previous year. Almost without exception the one thousand leading manufacturers who do 80 per cent of the total volume of business, have each year enjoyed increases. Under this condition not much thought has been given to the seriousness of inefficient methods, but in 1929 the situation changed. The volume of building business decreased and there was an important decrease in the output of concrete masonry as compared with 1928. Six hundred concrete products plants went out of business in the year. Profits dwindled and there developed an entirely different feeling toward the importance of manufacturing costs.

In about 1919 the concrete products business began to get into power production in earnest. To get away from the backyard type of plant seemed to be the all important job. The pendulum began its swing toward big, automatic production with large investments in land, buildings and equipment. Plants with one, two or even three hundred thousand dollars replaced those of three, five or ten thousand dollars in a period of from five to six years. The apparent or potential demand for the product did not justify the extent to which the construction of large plants has been carried.

The total sales of common masonry materials in the peak year, including common clay brick, clay tile, concrete block and sand-lime brick, were equivalent to approximately 1,250,000,000 (8 by 8 by 16) units. There are over 9,000 power concrete block, tile and brick machines installed and ready for operation, with outputs per nine hours varying from 1,000 to 3,000 per machine. Assuming an average output of 1,000 block per shift, these 9,000 machines can turn out, with properly designed plants, 9,000,000 units every nine hours. If these plants operate two nine-hour

shifts per day, they can produce in properly designed plants 18,000,000 block per day. If they operate 300 days per year, the output is 5,400,000,000 units or more than four times the total sales of all kinds of common masonry materials sold in any year. For the sake of conservatism, let's cut down the assumed very low average output per machine to one-half and then cut the number of working shifts in half and operate at only one quarter capacity, we still have an output of 1,350,000,000 block per year, which is 100,000,000 or 7 per cent more than the total sales of all types of common masonry combined.

As soon as profits began to fall leading manufacturers saw the seriousness of the gross over-capacity and corresponding over-investment. Consequently more manufacturers are studying efficiency this winter than ever before. Multiple shift operation is looked on by many of them as a possible means of lowering costs. Two hundred or three hundred products manufacturers with either entirely new or completely redesigned plants start into the concrete products business in an average year. Both groups are anxious to avoid the excess costs of over-capacity. Multiple shift operation is for new plants. It will not, for the most part, work in plants that are not designed for it.

Until a committee of the American Concrete Institute or some other interested organization makes a detailed study of costs of operating one shift a day in a plant designed for such operation and two shifts a day in a properly designed plant, it is probable that those who are responsible for plant design will draw upon the experiences of other industries and those who believe in multiple shift will so design their new plants and those who don't like the idea will continue as at present. Multiple shift operation, in my opinion, requires:

1. A plant designed for it and management favorable to it.

2. 20 to 30 per cent reduction in fixed capital investment, with resulting saving in interest and depreciation.

3. A lower cost of power, insurance and rent.

4. More capable supervision of men and plant, but not necessarily higher cost supervision.

5. Piece-work method of paying wages.

Mr. Wilk attaches much more importance to break-downs and inflexibility in small plants run at high capacity than I believe is warranted. The only reason we have break-downs is because we have machines and as the number of machines increases along with the interdependence of the machines, the greater the liability of break-downs. In operating a small plant two 9-hour shifts a day, taking out time for lunches, there still remain 4 or 5 hours for switching from one size of mold box to another, for changing tamper feet, mixer blades and replacing worn parts. Spare parts will have to be kept in stock. A foreman who is also the millwright will have to be on the alert for worn and poorly working parts, so that repairs will be made during off hours. Ninety per cent of the break-downs in a concrete products plant are due to carelessness and will be avoided in the efficient plant of the future.

Skilled workmen are usually not as efficient on night work as on day work, but concrete block plant workers are hardly to be rated as skilled. Efficiency in workmen on night shifts in concrete products plants is largely a matter of management. Care will have to be used in selecting the night workers, but assurance of steady employment will do much to overcome the difficulties of getting work done at night on an efficient basis.

The opinions expressed for and against multiple shift operation in particular and for greater plant efficiency in general for the time being are based on personal opinions and consequently are likely to be very pronounced in either direction.

To help the products manufacturer who is building his plant for the future, it is highly important that reliable information be made available which will enable him to reduce his costs to an absolute minimum and get output per plant much larger than the biggest plants are turning out at the present time with investments from one half to three quarters the present average.

Mr. Wilk: Before reading Mr. Parker's discussion, I want to comment briefly on Mr. Allan's discussion. If we are not to base a large part of the design of plants on the practical experience of the men who are operating them daily and to whom the capital invested in these plants looks for profit, then where are you going to look for advice on the design of plants? It goes without saying

that money invested in a plant is there to make a profit, and the men at the head of these plants are there to see to it that that investment brings a fair return. In the last ten years and particularly the last five years, considerable money and brains have been put into the business. If some of the largest manufacturers in the United States now come to the conclusion that single shift operation under present machinery conditions is best, then it strikes me that that practical opinion is very vital in the solution of the problem. Mr. Allan made one point about a market which did not have over-production. I dare say that in the United States there is not a market where there is not over-production.

As we go into walls above ground, we are going to have greater variety of units. When you have to manufacture special units for windows, etc., you have a different problem from that in the manufacture of units for basement walls. Instead of reducing the number of units, we are going to increase the number of units. Certainly we won't manufacture them all on one machine, because the number of changes you make in that machine will cost money.

I think, with Mr. Allan, that instead of having the 4,000 plants that we have today, we would be better off with 1,500.

Mr. Parker is engineer for one of the very largest products plants in the United States, at Ridgefield Park, New Jersey, supplying the northern part of New Jersey. They are manufacturing cinder units, and the machines in that plant are very large machines. He is looking at multiple shift from the standpoint of the large machine. I purposely took the small machine even though we do have large machines and small ones in our own plant. I want to give that background to Mr. Parker's paper, because he has not only the theoretical experience and training, but also the practical experience in a plant where he should have learned something about the value of single and multiple shift operations.

BY HENRY C. PARKER*

The answer to the question, "Should a Products Plant be Equipped for Single or Multiple Shift Operation," depends to

*Member Committee 707; discussion presented at 26th Annual Convention, in the author's absence by Benjamin Wilk, chairman of the committee.

some extent on the desired production, cost of land and equipment, methods of handling materials and ratio of maximum to minimum seasonal demands. The following estimates will show that a rather unusual combination of these conditions would have to exist to warrant multiple shift operation.

We are told that the tendency is toward the building of plants for large production. Let us assume a production of about 2,000,000 (8 in. or equivalent) units, made up of 4-in., 6-in., 8-in., 10-in., and 12-in. blocks. We will also assume, in this age of overproduction, a 5-day week. This would mean about 240 working days per year.

For a single or double shift plant, the required investments might be estimated as follows:

	Single Shift	Double Shift
Raw material handling	$ 6,000.00	$ 4,000.00
Mixing	7,000.00	4,000.00
Tamping	14,000.00	9,000.00
Pallets, Racks	9,000.00	9,000.00
Trucks	9,000.00	6,000.00
Building	22,000.00	20,000.00
Property & R. R. Siding	27,000.00	27,000.00
Total	$94,000.00	$79,000.00

These estimated amounts will vary greatly according to aggregate used, cost of land and railroad siding and to construction of building. The comparison between the two columns is less affected than their totals.

Let us consider separately the following general divisions into which cost may be divided:

(a) Material
(b) Labor
(c) Manufacturing Overhead, and
(d) Administrative Overhead and Selling

Of these, only the two items—Labor and Manufacturing Overhead—are affected by one or two shift operation of the plant. They represent about 40% of the total cost.

For low labor cost, the one shift plant is best. In addition to the several reasons given by Mr. Wilk, we find that it is much

easier to keep all employees busy on large production from single shift operation, than on small production. The operation of a large modern plant is similar to a complicated machine. Each man has his special work to perform and often he cannot do two different operations, even though he is not busy all of the time on his allotted task.

For example, one lift truck would be kept busy taking green blocks to the curing room on a production of 9,000 blocks per day, and a second truck would have all it could do to take the same quantity of blocks from the curing room to the storage yard. On half that production neither truck would be busy, while one truck could not do both operations.

The difference in labor cost between one and two shift operation could be accurately estimated for a given production and plant layout, but the amount arrived at would not apply to a different production or plant layout.

Manufacturing overhead is composed of many items, such as, depreciation of equipment, depreciation of buildings, repair parts and labor; superintendence; non-productive labor; bookkeeper; compensation insurance; fire insurance; interest on investment; taxes; power, heat and light; etc.

Of these, the single shift would decrease repair parts and labor; superintendence; power, heat and light; and the two shifts would decrease fire insurance; interest on investment; taxes. The other items remain about the same for either one or two shift operation.

With one shift operation, there is ample time to keep the machinery in good condition, thus lessening repairs; one superintendent can care for the entire production, and assistant for night work is not required. To light the plant adequately for all operations from unloading materials to storing blocks in yard, would consume a large amount of current.

Fire insurance, interest on investment, and taxes, would show a saving on the two shift plant assumed above of about $2,000 per year, not enough to pay the night foreman.

A word about 12-month operation might be of interest.

To insure shipment of cured products, at least 30 days production should be carried in stock. This amounts to 270,000 on the

assumed output above of 9,000 per day. At a cost of ten cents in the yard, this stock represents $27,000, and a fixed charge of $1,620.00 per year.

If a plant were operated nine months of the year and winter shipments averaged 3,000 per day, reserve stock would have to be increased by 180,000 blocks carried for an average of one and one-half months and increasing the fixed charge on money invested in finished product by only $135.00. Theoretically, this would appear to be an excellent method. But when consideration is given to the injustice of laying off all employees engaged in manufacture for three winter months and the cost of getting together and training a crew of new men in the spring, a twelve month year is far better.

A ten hour day is a thing of the past and needs no consideration here. The necessity of cleaning equipment after each day's run is a good reason for running nine hours instead of eight hours per day, and five days instead of five and one-half days per week.

There are many good reasons for building and equipping a plant for an average one shift production to cover the year's shipments, stocking up in the winter for the spring rush, and making provision for the addition of equipment, mainly pallets, racks and curing rooms for a second shift, if necessary. Planning for a five day week also permits increasing production 20% by working a six day week without adding to equipment.

Mr. Wilk—Discussion of unemployment seems to indicate that the men at the head of large plants are giving serious consideration to the question of unemployment. In Detroit the large automobile plants did the bulk of their manufacturing for the year by September and then laid off men. That had a great deal to do with the large amount of unemployment there, but they are giving that matter very serious consideration now.

Charles VanHouten (Chicago, Ill.)—We tried the long hour day in the manufacturing of block in Chicago, in two ways. Our first way of trying it was starting the men at work at six o'clock in the morning. Some of these men were to work eight hours, and the better and more physically fit were kept 12 hours, and when these other men were let out, the men to take their place of course

stepped in. That worked very successfully. We got as good production during the last part of the day as the first part. We started out that way because we thought our business would not require greater production. We worked about two months in that manner and changed to two entire shifts, working each gang eight or nine hours. That proved to be an absolute flop. We tried to get as much production, we had the same men doing the work at night as we had had during the day, but it was a failure. Men would not stick to the job.

Discussion of Paper by P. M. Woodworth

"Some Tests of Concrete Masonry Cured with High Pressure Steam"*

BY W. H. CRUME†

At the time of making the tests of concrete units cured by high pressure steam, in charge of Mr. P. M. Woodworth, we were making only vibrated units. The results were so far superior to any other method of curing that we have adopted this practice for all our concrete units.

We are manufacturing various sizes of products on tamping machines both of sand and gravel and of Haydite.

With the vibrated units we have improved the process and equipment and now use leaner mixtures, a 1:6:8 mix by volume gives us a yield of better than 100 of 4x5x12 in. units per bag of cement, the units testing 1500 to 1800 lbs. per sq. in.

We have recently begun producing concrete brick, using only portland cement and our bank sand. In this department we follow the same practice as in sand-lime brick making, using only 4 sacks of cement per 1,000 standard size brick. The cement and sand are measured by poidometers and fed to a revolving rod mill of 4 ft. diameter by 10 ft. length. In this mill the materials are ground and mixed by 7 tons of round rods rolling and tumbling, so the sand grains are well coated with cement, the water being added in the mill.

The concrete brick, 2¼x3¾x8 in. and double brick 5x3¾x8 in. are formed in presses and are handled in their green state without pallets being stacked on cars that are then run into high pressure steam cylinders. When removed they are shipped direct to the job, in fact all our concrete products are ready to use with no further curing.

*Journal, A. C. I., Feb. 1930; *Proceedings*, Vol. 26, p. 504.

†President, The Crume Brick Co., Dayton, O., in whose plant the tests reported by Mr. Woodworth, were made.

It takes about three hours to heat the large cylinders and we find that 6 hours at 100 lbs. pressure is sufficient. It takes about one hour to exhaust the steam. Thus the cycle is 10 hours, using the curing facilities twice in 24 hours.

In addition to high early strength and practical elimination of shrinkage it is not necessary to have large inventories of products in the curing yard; there is very little breakage, as the products are hard and tough.

The color of our vibrated tile and pressed brick after coming from the high pressure steam is almost white, even though made from standard grey cement and brownish bank run sand gravel. In fact the use of white cement does not make them any lighter in color.

We have recently experimented in mottled color effects with the vibrated tile and have obtained some beautiful effects, with clear whites and brilliant colors in reds, orange, greens, browns, blacks, etc.

We have tried these colored units with low pressure steam, but colors show up dull and the background muddy.

We feel that there are great possibilities in this method of curing and that we have just scratched the surface.

BY L. J. POND*

BEFORE a manufacturer departs from well accepted methods of curing at an added capital expense and increased cost of running extra plant there should be no doubt as to the advantages to be gained by the outlay. I submit that case for high pressure steam curing is not proven.

That in view of development of rapid hardening cements now almost universal in Great Britain, and soon to be so all over the world, the advantage of early handling of units can be more cheaply obtained by using this rapid hardening cement without forcing the curing by steam, either high pressure or otherwise, and this without any special plant.

Before data can be obtained as to durability of units cured by high pressure steam over a period of years, rapid hardening

*Managing Director, Craighall Cast Stone Co., Ltd., Edinburgh, Scotland.

cements will be available everywhere, and steam curing plants and investigations will be obsolete.

Decrease of compressive strength of steam cured units at nine days indicates great internal stresses that are probably closely related to subsequent surface cracking.

Panel test conditions do not correspond with severe weather conditions in a damp climate having great variations. I suggest that these are: Constant alternations of prolonged rain, say three days with wind pressure soaking it to maximum absorption, followed immediately by rapid drying out of one surface by drying winds and sun while the body of the unit is still full of absorbed water. These conditions set up severe stresses within the unit and are believed to have a large part in forming cracks in the surface, and thus the commencement of the breaking-down process by frost and further moisture ingress.

Further tests of high pressure steam cured units under these conditions would be of great value to ascertain if such units possess any advantage over others cured by more common means.

Readers are referred to the JOURNAL *for November, 1930 for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1930.*

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

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MATERIALS

ADMIXTURES

The chemical behaviour of siliceous materials, trass and diatomaceous earth in lime and cement mortar. P. MECKE. *Tonind. Ztg.* (Germany). March 30, 1930, V. 54, No. 26, p. 444-6.—Combination between lime and siliceous materials appears to be very slight in lime and cement mortars, unless lime soln. remains in contact with them for a long time.—F. O. ANDEREGG

The effect of mixing time and of trass addition on the compressive strengths of fluid mortars. (See PROPERTIES OF CONCRETE)

The calculation of sieve analyses and the fineness modulus by Abrams. A. HUMMEL. *Zement* (Germany), April 1930, V. 19, No. 15, p. 355-64.—Way is shown to correct valuation of aggregates by means of sieve analysis. Density and surface of aggregates are of fundamental importance for concrete strength while gradation of coarse aggregates has little influence. Comparison of sieve curves with ideal curves gives much information. Water requirement of concrete mixes is direct function of gradation. Fineness modulus, which can be calculated from sieve analysis, allows a direct valuation of aggregates regardless of sieve curve. This relation is illustrated by several synthetic samples and samples of practice.—A. E. BEITLICH

Report of Committee on Standard Specifications of the National Sand and Gravel Association. STEPHEN STEPANIAN. *National Sand Gravel Bull.*, March, 1930, V. 11, No. 3, p. 14-20.—Work of committee, organized in 1926, has been directed principally toward selection of commercial sizes suitable for use as basis for production and for specifications. In preparing specifications for grading, committee took into consideration comparatively wide range in grading of material encountered in different localities. Committee formulated recommendations of extreme limits within which it was felt that sand and gravel, satisfactory for usual purposes, should fall, with thought that specifications with narrower limitations and falling within the recommended over-all limits should be prepared in each locality to meet needs of local conditions and insure product of proper uniformity. Recommendations for commercial sizes of gravel, were stated in terms of both square and round openings, square openings to govern in case of dispute. Material with not more than 5 per cent coarser than the maximum or 10 per cent finer than the minimum size is to be considered as conforming to these requirements. In addition to proposal for commercial sizes of gravel, Committee reported on its recommendations for sieve sizes, specifications for grading of sand for inside plaster, and specifications for grading of various proposed commercial sizes of gravel.—P. MCKIM

A "straight - line" designed crushed stone plant. *Rock Products*, March 29, 1930, V. 33, No. 7, p. 57-61.—New plant of Federal Crushed Stone Corp. seven miles east of downtown Buffalo, N. Y., is equipped with 42-in. gyratory crusher, 10-in. Newhouse crusher, vibrating screens for both scalping and final sizing, and concrete stave silos for loading bins. No elevators are used, all conveying being done by belt. Plant of simple straight-line design, scalping and recrushing being done at ground level, and only finished sizes going to screen houses. Designed capacity is 300 tons. Dump trucks are used for quarry transport. Quarry equipment comprises wagon drills, jackhammers, compressors, 5-yd. steam shovel, 5 motor trucks and 1½-yd. electric shovel for stripping. Screening system is in duplicate. A 25-ton gas locomotive is used for railroad car switching. Plant is operated with crew of 30 men. Ten illustrations, plan and two elevations. —JAY BOHN

New Diesel - powered crushing plant in Kentucky. *Rock Products*, March 1, 1930, V. 33, No. 5, p. 41-46.—

Sunbeam Quarries Co., Clermont, Ky., put largest crushing plant in state in operation in September, 1929, having capacity of 1500 tons of crushed stone per day. Caterpillar "60" and trailer purchased for stripping were used temporarily for quarry transport and proved so satisfactory that this system will be employed. Quarry equipment includes gas-electric shovel, wagon drills and jackhammers. Plant was designed and equipped by Allis-Chalmers Mfg. Co. Power furnished by 560 H. P. diesel engine. Eleven illustrations; four plans and elevations. —JAY BOHN

Investigations concerning concrete aggregates. (See PROPERTIES OF CONCRETE)

Effect of characteristics of coarse aggregates on the quality of concrete. See PROPERTIES OF CONCRETE)

CEMENT

Raw mix control in cement manufacture. G. W. JORDAN. *Rock Products*, March 29, 1930, V. 33, No. 7, p. 55-56—An endeavor is made to show that essential factor in proportioning raw materials is not calcium carbonate content, or total carbonates, not "hydraulic modulus" or "molecular ratio" but lime-silica index; that all other things being equal, correct value for this index will increase as the $\frac{SiO_2}{R_2O_3}$ ratio decreases. It cannot be too strongly emphasized that of equal importance to chemical composition in determining correct lime-silica index are the physical factors, fineness to which the raw materials are ground, and temperatures and duration of burning. Due to research work accomplished in past few years, several controversial points are cleared up. Essential compounds in portland cement can now be definitely set forth as: tricalcium silicate ($3CaO \cdot SiO_2$), dicalcium silicate ($2CaO \cdot SiO_2$), tricalcium aluminate ($3CaO \cdot Al_2O_3$), and a compound represented by the formula $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ with the MgO remaining uncombined.—JAY BOHN

Fine grinding of portland cement materials. ANTON J. BLANK. *Rock Products*, March 1, 1930, V. 33, No. 5, p. 64-65.—Various results are obtained in way of output and power consumed by use of various sized grinding media in loading of tube mills. Through changes, certain raw grinding mills were loaded with 1½-in. round balls in place of customary load of ⅞ to 1¼-in. round balls. Output was 600 kilos per hour less, and power consumption was 2.1 kw. greater per metric ton of output. Summary of test carried on with finish grinding tube mills indicated output averaged 1000 kilos less per mill hour, and power consumption was 5.4 kw.h. greater per ton of cement ground, accompanied by slight drop in fineness. Tabulated results of nine carefully recorded and comprehensive tests. Other incidental notations on results of these tests.—JAY BOHN

The present state of the method of filtration of cement slurry. C. H. SONNTAG. *Zement* (Germany), March 1930, V. 19, No. 13, p. 302-4.—Description of Oliver Feinc and an American filter and their application for filtration of slurry of following raw materials: natural cement rock, limestone and slate, limestone and clay, marl and clay and cement dust. Advantages of filtration in cement manufacture are discussed and cost calculation for this installation is given. —A. E. BEITLICH

The effect of ashes and flue dust on cement. A. WEISSMANN. *Tonind. Ztg.* (Germany), Apr. 24, 1930, V. 54, No. 33, p. 563.—Certain variations in strength in portland cement coming from wet process kiln were ascribed to variable content of coal ashes or flue dust in finished cement. These dusts tend to accumulate at center of larger clinker balls and are also found in finest clinker materials. Trouble was overcome by adding flue dust to raw clay.—F. O. ANDEREGG

Holland's first and only cement plant. *Rock Products*, March 1, 1930, V. 33, No. 5, p. 58-63.—Plant was built in St. Pietersberg hills near Maastricht, to take advantage of large chalk deposits and Maas River and

canal transportation. Project was developed by "Erste Nederlandische Cement Industrie," (Enci), the plant having initial capacity of 6000 bbl. per day. "Solo" principle (combination of number of operations into one machine) and large units have been used. Limited plant area between chalk deposits and Maas canal necessitated plant length of 1200 ft. with little depth. Chalk is loaded by steam shovel to cars and delivered to two-roll crusher. Crushed chalk is belt conveyed to wash mill department and bins. One conveyor is movable and reversible. Clay from ships or storage is washed and pumped to chalk wash mills affording preliminary check of mixture. Raw slurry is collected from wash mills and pumped to blending tanks, agitated by compressed air. From there it is pumped to tank feeding three wet grinding three-compartment solo mills. Wet mills discharge to 6 slurry tanks. Slurry is pumped 750 ft. to kiln house. Four rotary solo kilns are fired with pulverized coal from movable 40 ft. nozzle. Flue dust is electrically precipitated. Clinker is crushed, elevated and stored for standard portland cement clinker and for "super cement." Five solo mills are used for finish grinding. Finished cement is pumped from storage to pack house containing 4 two-tube packers, bag cleaning machinery and dust collecting system. Testing and control laboratory, machine shop, transformer substation, compressor room and business office are established at plant. Plant layout designed for expansion by future addition of raw mills, kilns, clinker grinding mills. Entire design and machinery installation by G. Polysius, A. G., Dessau, Germany, which company furnished practically all equipment. Twenty-one illustrations; one plan.—JAY BOHN

The quality of air separated cements. A. B. HELBIG. *Rev. materiaux construction trav. publics.* (France) Feb., 1930, No. 245, p. 76-8.—In the past it has been generally admitted that air separated cements have less strength than those from standard tube mills, but methods have now been worked out where yield is increased with no reduction in quality.—F. O. ANDEREGG

Aluminous cement. HAEGERMANN. *Tonind. Ztg.* (Germany). March, 1930, V. 54, No. 23, p. 393-6. It is suggested that high early strength cements can be made to serve almost all purposes instead of higher priced aluminous cement made from imported bauxite. Some of properties of latter, including special manipulation required and variable composition, are brought out.—F. O. ANDEREGG

The use of bauxite in manufacturing portland cement. J. DEFARGE. *Rev. materiaux construction trav. publics.* (France) Jan., 1930. No. 244, p. 11-13, Feb., No. 245, p. 64-7.—A siliceous limestone, low in iron oxide and alumina, did not burn properly even on adding iron ore. Use of bauxite containing both oxides solved problem. Author prefers wet process and describes a plant employing it.—F. O. ANDEREGG

Latest developments in the Japanese cement industry. MITZUZO FUJII. *Rev. materiaux construction trav. publics.* (France). March, 1930, No. 286, p. 70-3.—Statistics are given showing the present favorable position of cement industry in Japan. Their chief raw materials are limestone and argillaceous schists.—F. O. ANDEREGG

The cement modulus. HANS KUHL. *Tonind. Ztg.* (Germany), March, 1930, V. 54, No. 23, p. 389-92. It is concluded that the more completely lime is brought into combination with silica, iron and alumina the better the cement. Therefore, proposal is made that hydrauiic modulus be dropped from standard specifications for portland cement and replaced by degree of saturation with lime.—F. O. ANDEREGG

The development of the separating partition in the compound mill. CARL NASKE. *Tonind. Ztg.* (Germany), March 20, 1930, V. 54, No. 23, p. 386-9.—Description of various types of partitions which permit properly ground material to pass into the next compartment in cement manufacturing process.—F. O. ANDEREGG

The reduction of heat losses of wet process rotary kilns. HANS ENGEL. *Tonind. Ztg.* (Germany), March, 1930, V. 54, No. 24, p. 408-9.—Thickening or filtering of slurry is helpful in increasing efficiency of rotary kiln used in wet process.—F. O. ANDEREGG

About the setting of cement. "Ueber das Abbinden des Zements." Report No. 41 of the Eidgenoessisches Materialpruefungsamt der Techn. Hochschule in Zuerich. HERMANN GESSNER. 1929. Th. Steinkopff, Leipsig. R. M. 4.00. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Feb., 1930, V. 82, No. 9-10, p. 81.—The book describes studies on the viscosity and electrical conductivity of cement-water mixtures. Experiments are made concerning measurements of hydration and swelling of cement. (cf. *Kolloid Zeitschr.* (Germany), Nov. 1928, V. 46, No. 3, p. 207-16; Jan., 1929, V. 47, No. 1, p. 65-76 and Feb. 1929, V. 47, No. 2, p. 160-76.)—A. E. BEITLICH

The construction of the new cement plant in Stupava (Czecho-Slovak Republic). WILHELM PETRY. *Zement* (Germany), March 1930, V. 19, No. 12, p. 277-9.—Construction was started in September 1929. Plant is based on wet process and built for output of 7000 carloads per year. Enlargements are possible to increase this figure up to 14,000 carloads. Raw materials, clay and limestone, are transported by aerial tramway. Slurry tanks, kiln and coal mill are in one 136 m. long building. Iron structure of the roof rests on 20 m. high reinforced concrete columns. Eight cement storage bins are 3.5 m. by 14 m. each and 15 m. high. Sliding forms were used for their construction. Most of the buildings were built with regular and high early strength cements; alumina cement was used for slurry tanks.—A. E. BEITLICH

The appearance of cracks at the edges of cement pats during storage in air after water storage. H. BURCHARTZ. *Zement* (Germany), March 1930, V. 19, No. 12, p. 265-6.—Test pats sometimes show fine hair cracks and bigger cracks running from the edge to center, when they are stored in air after normal water storage. These cracks are shrinkage cracks and should not be mistaken for cracks in unsound cement. Tests were made to find out how many cracks appear under different storage conditions and temperatures. On account of these shrinkage cracks, the cement pats should not be kept in air longer than one hour after water storage, before they are examined for unsoundness.—A. E. BEITLICH

Rotary clinker waste heat and combustion air. Report of the furnace committee of the Verein Deutscher Portland Cement Fabrikanten. *Zement* (Germany), March 1930, V. 19, No. 12, p. 287-91.—Recovery of heat by preheating combustion air in per cent of total heat consumption and the temperature of the combustion air which can be reached by such preheating is shown. Curves are given for accurate calculations of amount of air in case of different heat consumptions and heat content of this air at different temperatures. Several references are given.—A. E. BEITLICH

Colloidal cement hardening I. HANS KÜHL, DR. GOTTFRIED AND DR. THILO. *Zement* (Germany), March 1930, V. 19, No. 12, p. 262-4.—Hardening of cement is process of hydration, mechanism of which is not completely worked out. Two extreme schools of thought exist. One holds that hardening is entirely due to formation and interlocking of crystals of silicates and aluminates of lime, similar to hardening of gypsum. Other school holds that hardening process is formation of colloidal gels which cement together. Author tries to prove this latter theory by preparing two synthetic slags, fused of pure raw materials in oxy - acetylene flame. Microscopic and X-ray examination showed that one slag was completely amorphous, while other one contained only traces of crystalline substance. These samples were mixed with solution of potassium hydroxide (free from carbonate) and strength tests were made. Setting properties were normal.

Slag which was absolutely amorphous gave with 10 per cent mixing solution 8.2 kg./cm² and with 20 per cent 9.6 kg./cm² after 3 days. Slag with traces of crystalline material gave with 10 per cent mixing solution 23.6 kg./cm² after 3 days. Hardened samples showed under microscope same absence or presence of crystalline material as original samples. X-ray studies did not detect any traces of crystals in the first slag which fact proves, that a hardening process is possible without any crystalline phase.—A. E. BEITLICH

Recent developments in the application of closed circuit fine grinding. A. ANABLE. *Rock Products*, March 29, 1930, V. 33, No. 7, p. 66-69.—Comparison of metallurgical and cement industries shows that cement manufacturers have not profitably applied experience gained by metallurgical plants with regards to closed circuit fine grinding. Based on this experience closed circuit grinding should prove of value in wet process cement manufacture to following extent: (1) Closed grinding should increase mill capacity 50 per cent; (2) should reduce wear on mill liners and grinding media 25 per cent; (3) should produce consistently uniform 200-mesh product. Recent investigations show there is considerable room for improvement in cement raw grinding practice. In metallurgical practice every attempt is made to remove primary slime at outset, and finished material from and between each stage of grinding as soon as reduced to fineness intended for that particular stage. Average of eight western copper plants show power consumption of 9.74 kw.h. per ton for 100-mesh, and 12.25 for 200-mesh. Certain physical, thermal and chemical conditions are essential for correct burning, especially for high early strength cement. The conditions are: Fineness, temperature, time of contact in zone of maximum temperature, chemical composition, and physical structure. Closed circuit grinding was expected to prove helpful in cement industry for reasons: (1) Definite control of fineness; (2) better mixing of raw material before burning; (3) better grinding economy through reduced power consumption and smaller mills. Laboratory investigations of these studies showed that superfines of "flour" were not as important as usually supposed, but that small amount of oversize material coarser than critical mesh was largely responsible for incomplete combustion, unsoundness due to free CaO, and necessity of high temperatures to counteract effect of oversize. Tables give comparative results.—JAY BOHN

Calculation of raw mix for experimental production of clinker. HAROLD H. STEINOUR AND HUBERT WOODS. *Rock Products*, March 15, 1930, V. 33, No. 6, p. 75-76.—In study of effect of systematic variations in the chemical composition of cement raw mix, desired compositions may be obtained by different ways. If chemically pure oxides are used, their proportioning is simple matter, but in any quantity, rather costly. Chemicals of technical purity may be used as their impurities are simply other oxides normally present in cement mix. Differences between laboratory and plant mixes may be partially overcome if plant kiln feed is used as basis for new compositions, desired changes being obtained by addition of amounts of oxides necessary. In table compiled on composition of raw materials, symbols for ignited materials are: plant raw mix, R; Calcium Carbonate, C; Magnesium Carbonate, M; Silica, S; Alumina, A; and Ferric oxide, F. Table gives equation:

$$CaO = 0.969C + 0.036M + 0.018S + 0.0A + 0.0F + 0.65R$$

For general solution CaO and R must be treated as constants of unassigned values. By transposition similar equations may be set up for remaining oxides. All clinkers prepared in tests were analyzed and results could normally be counted on to check calculated values within 0.5 per cent.—JAY BOHN

Volume constancy of cements. (See PROPERTIES OF CONCRETE)

The determination of the hydrogen ion concentration as an aid in cement and concrete research. (See PROPERTIES OF CONCRETE)

MISCELLANEOUS

About the relation between particle size and voids in products consisting of loose particles. A. H. W. ANDREASEN AND J. ANDERSEN. *Kolloid Zeitschr.* (Germany), March 1930, V. 50, No. 3, p. 217-28.—Based on theoretical deductions it was found, that every product which consists of loose particles, being present without any order, has a certain relation between these particles and the voids. When coarser particles are added to the first product the equation: C (K) = K_q must stay constant if the relation between particles and voids should not vary. Particle sizes and percentages are expressed by the values of C and q; K is a constant. Experiments were made with two and three different gradations observing directions by Gum and Kerkhof to obtain a product as dense as possible. Products were made with variations of q by means of Fuller-curve and voids were compared. In all examinations the voids were measured for loose filled and for packed materials. It is shown, that it is not possible to obtain most dense products with few gradations. This can only be done when all particle sizes are present within certain limits.—A. E. BEITLICH

Absorbing properties of silica gels. A. P. OKATOV. *Journal appl. chem.* (Russia), 1929, No. 2, p. 21; reviewed in *Industrie Chimique* (France) April 1930, V. 17, No. 195, p. 241.—Absorbing properties of silica gels are independent of coagulation temperature of solution and concentration of silica in this solution. Below 25° C. it is difficult to obtain solution containing more than 12.5 per cent SiO_2. Experience has shown that presence of monobasic acid increases speed of coagulation while presence of NH_3 seems to retard this action.—A. E. BEITLICH

PROPERTIES OF CONCRETE

Tests on the behavior of cement mortar in hot water. OTTO GRAF. *Deuts. Ausschuss für Eisenbeton*, (Germany). Heft. 62, 1930, *Bldg. Science Abstracts* (England), March, 1930.—Tests were made as study of effect on concrete strength produced by high temperatures in hot water tanks, chimneys, etc. Tensile and compressive tests of 1:3 and 1:6 mortars were made. Various types of storage, using temperatures of 20°, 50° and 90° C. were employed. Effect of storage in water at 90° C. was not great for 1:3 mortar, giving some reduction in strength at greater ages, as compared to 20° C. storage. The 1:6 mortar showed distinct increase in strength in similar tests with 90° C. storage. Effect of storage in water at 50° C. was not consistent for different cements used. Curing in water vapor gave higher tensile strengths than did water curing, at 50° temperature. Rapid alternations from 90° to 20° C. produced strains that led to development of cracks.—F. E. RICHART

Investigations concerning concrete aggregates. GILBERT NESS. *Str. Eng.* (England), April, 1930, V. 8, No. 4, p. 166-172.—Six-inch cubes, made with various fine and coarse aggregates and cement, were tested at ages varying from 2 days to 12 months. Compressive strengths developed and portion of coarse aggregate sheared were used to measure value of aggregate used. It was concluded that quality of coarse aggreagte was not nearly so important as that of the matrix and that coarse aggregates containing soft particles may be used when concrete is not to be stressed above 600 lb. per sq. in. and is not to be used in structures subject to water pressures or exposed to weather without waterproofing.—V. P. JENSEN

Rapid-hardening portland cements. E. O. WILLIAMS. *Conc. Constr. Eng.* (England), March, 1930, V. 25, No. 3, p. 182-183.—Compression and bond tests were made with ordinary and rapid-hardening portland

cements. Compression tests on 6 in. cubes; bond tests with polished bars embedded axially at each end of concrete blocks 6 by 6 by 12 in., forming double pullout specimen. Concretes were of 1:2:4 and 1:3:6 mixes. Compression tests showed strength of 1:2:4 concrete made with rapid hardening cement at 7 days to be 2½ to 3½ times that of ordinary concrete at 21 days. With 1:1:2 concrete somewhat less advantage was found. Bond test results were erratic, but indicated that bond with rapid-hardening cement at 7 days exceeded that for ordinary cement at 21 days.—F. E. RICHART

The effect of mixing time and of trass addition on the compressive strengths of fluid cement mortars. A. STEOPPOE. *Tonind. Ztg.* (Germany) May 1, 1930, V. 54, No. 35, p. 590-1.—The mix was of equal weights of cement and sand with 25 per cent water. Elec. stirring was used at 100 R. P. M. for 5, 80 and 170 min. The densest mortar was obtained by mixing for 80 min. The same was true for neat cements. A standard and a high early strength portland cement gave highest strengths after 80 min. mixing, while a Kuhl cement showed diminutions on continued mixing.—F. O. ANDEREGG

Tests of pavement concrete being conducted by the U. S. Bureau of Public Roads. F. H. JACKSON, *Crushed Stone J.*, April, 1930, V. 6, No. 4, p. 3-9.—Essential characteristics of concrete for highways are durability and strength, and transverse strength seems more important than compressive strength. Freezing tests indicate that when water-ratio exceeds 1.0, durability of concrete is controlled by durability of mortar and that low water-ratio mixtures may require aggregate of greater durability than those having high water-ratio. Coarse aggregates have particularly important influence on transverse strength and therefore given water-ratio does not insure given modulus of rupture. High and low strengths are obtained within different classes of aggregates so each individual aggregate must be considered as individual case. New specification is proposed based on transverse strength. In its operation, preliminary tests are made to determine correct proportions for desired transverse strength with particular aggregates available for job. As result of these tests contractor is supplied with batch weights, or weights of cement, sand and coarse aggregate per cubic yard of concrete. Specification also covers in detail desired characteristics of concrete ingredients.—A. T. GOLDBECK

Ready-mixed conctete. STANTON WALKER. *National Sand Gravel Bull.* February, 1930, V. 11, No. 2, p. 54-58.—From a study of available information on the ready-mixed concrete industry, following conclusions have been drawn: (1) Ready-mixed concrete industry gives promise of becoming stabilized business and therefore does not offer prospects of more than conservative return on investment. (2) Field for ready-mixed concrete is broad enough to include all types of concrete construction and opportunities for development in this direction are unlimited. (3) Only careful study of actual or potential building program of community will determine whether it can support ready-mixed concrete plant. Generally speaking, community with population of 50,000 should, under favorable conditions, permit successful operation of ready-mixed concrete plant. (4) Availability of raw materials is first importance in location of ready-mix plant. (5) Measured by ordinary business experience, gross sales of ready-mixed concrete plant must be relatively large as compared with investment. (6) Question of proper equipment for ready-mixed concrete plant is matter which can be determined only by consideration of local conditions. (7) Tests have shown ready-mixed concrete can be transported for any reasonable distance without deterioration in quality of concrete by use of equipment now available. (8) Ready-mixed concrete operator is ideally situated to exercise equivalent of laboratory control over concrete used in ordinary construction.—P. MCKIM

Effect of characteristics of coarse aggregates on the quality of con-

crete. STANTON WALKER. *Bull. 5*, National Sand and Gravel Association. March, 1930, 16 pages. The more authoritative conclusions which have been drawn from available information are outlined, suggestions are made concerning factors on which data are needed and some features are cited which have made development of exact information slow and difficult. More important factors which have direct bearing on quality of concrete are summarized and conclusions drawn from study of various data outlined are: (1) Aggregate characteristics have important effects on quality of concrete, but these are generally of less importance than effects which may result from variations in proportioning, mixing, placing and curing concrete. (2) Aggregate characteristics which affect quality of concrete are, in general, common to different types of materials. Studies of them should be made without reference to type. (3) Studies of aggregate characteristics should include consideration of significance of durability, hardness and strength of aggregate particles, surface texture, shape, deleterious substances and grading. (4) Except for durability and certain deleterious substances, there are, in general, no significant aggregate characteristics the effect of which cannot be compensated for by relatively minor changes in proportions. (5) Indications of few tests available are that aggregates which are unsound for given condition of exposure will cause eventual disintegration of concrete exposed to same conditions. (6) Carefully controlled tests of different aggregates falling within range of quality of those commonly used for concrete have shown differences in transverse strength of concrete of approximately 25 per cent due to aggregate characteristics other than grading, soundness, or deleterious substances. Rough estimate would place responsibility on strength of particle for about one-half of this difference and one-half on surface texture. (7) Principal effect of more common so-called deleterious substances, such as soft and friable particles, is on appearance of concrete for comparatively wide range in quantity present. (8) Variations in grading of coarse aggregates cause variations in economy, workability, yield and strength of concrete.—P. MCKIM

Concrete for resisting sea water. H. E. SQUIRE. *Contr. Jour.* (England) Apr. 16, 1930, V. 102, No. 2653, 2654, p. 1265, 1345. (cf. *Proceedings* Am. Con. Inst., 1929, V. 25, p. 751-6.) Marine structures deteriorate from mechanical abuse of sea water and its chemical effects on concrete. Deterioration manifests itself first, by softening, cracking and spalling of surfaces, normally attributed to chemical changes and leaching out of slightly soluble ingredients, such as calcium hydrate; second by spalling of concrete due to corrosion and expansion of steel reinforcement. First type occurs below water line and depends on design and quality of concrete which should be uniform and capable of resisting absorption and circulation of water. Concrete with high compressive strength is usually obtained with a low water content and is not therefore necessarily most suitable for sea work on account of great absorption. Reasonably high cement content, well graded aggregate and uniform quality are essential to satisfactory concrete. Sufficient water should be used to produce a mushy consistency, with slump of 3 to 5 in. and 28-day strength of 2500 lb. per sq. in. Aggregates should be inert and non-absorbent. Washed and screened gravels are preferable to crushed and dry screened rocks owing to varying accumulation of rock dust in latter. An addition of fixed quantity of rock dust, fine sand or diatomaceous earth to produce stickiness and greater uniformity may, however, be beneficial. Hydrated lime should not be used. Tamping, after mixing, is as essential to good concrete as careful proportioning of materials, and sufficient labor should be provided to handle required output without undue haste. Precasting members is much preferable to casting concrete under water because of greater uniformity of product obtainable. Increasing fineness of cement and limiting magnesia content to 3 per cent were not proved essential. Steel needs cover of 4 to 5 in., even with good concrete, to prevent corrosion, and obtain protection. Damage due to

such corrosion appears usually as an occasional crack after 8 or 10 years, but does not seriously affect stability under 20 or 25 years, the life of reinforced concrete structure as ordinarily designed. Present value of fund to replace structure in 25 years, is less than 30 per cent of first cost, based on 5 per cent interest, while absolute permanence is worth an additional first cost of about 42 per cent. Life of structure is prolonged by protecting steel. One expedient is to give greater cover than ordinarily employed, and to use small diameter bars which exert less pressure. Angles and heavy concentration of steel in beams offer less resistance than smooth surfaces and slabs. Impact from ships should be carried axially on piles rather than by bending strain in structure. Asphaltic coatings with sufficient viscosity to heal over crack should be employed to prevent deterioration. Injection of asphalt under heat and pressure is expensive and not applicable to deck work. Metallic coatings to steel reduce the bond slightly, but galvanizing has so far proved successful. Cadmium may also prove successful, and both zinc and cadmium are electro-positive to iron and thus retard corrosion. It is more probable that permanence in reinforced concrete as structural material for marine work, may be obtained by surface coating or metallic coating to steel rather than fro m protection from the concrete itself.—JOHN E. ADAMS

Improvement of concrete masses through a vacuum process. *Das Betonwerk* (Germany), March 23, 1930, V. 18, No. 13.—Just as it is possible to produce plaster or concrete masses with excess of pores by employment of rarefied air, it is also possible to produce a dense concrete by extracting air from concrete mass during mixing operation. Machines are now being perfected to bring theoretical possibilities into practice.—H. FRAUENFELDER

Volume constancy of cements. HANS KUHL. *Tonind. Ztg.* (Germany) Feb. 27, 1930, V. 54, No. 17, p. 278-81. —A discussion of the retrogression in strength which often follows the wetting of dry concrete, colloidal properties of the cement and their volume constancy.—F. O. ANDEREGG

Mixing water as a cause of a setting failure. G. HAEGERMANN. *Zement* (Germany), March 1930, V. 19, No. 12, p. 264-5.—Two concrete floors, made of same aggregate and cement showed different behavior on the job. One was satisfactory but other failed, concrete did not set and heavy coating of calcium carbonate crystals formed on surface. Chemical and physical tests showed uniformity of raw materials, but concrete which failed contained up to 2 per cent N_2O_5 and P. These impurities came from mixing water which was not sufficiently purified when second floor was placed and contained 4.5 per cent $N_2 O_5$. An amount of 0.75 per cent $N_2 O_5$, was without any effect on the setting process.—A. E. BEITLICH

The determination of the hydrogen ion concentration as an aid in cement and concrete research. KARL BIEHL. *Zement* (Germany), March 1930, V. 19, No. 12, p. 269-73. —Method is given and apparatus described to determine hydrogen ion concentration of waters, which might have a corroding effect on concrete. Water containing carbon dioxide deteriorates concrete, made from ordinary portland cement, only, when its hydrogen ion concentration is 7 or below 7. Tables and curves are given showing the amounts of free and combined carbon dioxide in the water and their corresponding p_H. Additional tests showed the influence of salts, dissolved in the water, on the p_H. Second part of article deals with determination of solubility of 16 different cement brands, portland cements, made in rotary and shaft kilns, blast furnace slag cements, Eisenportland cements, high alumina cements and mixed cements. The alkalinity and hydrogen ion concentration were determined in order to find a means to measure the resistance of cements against corrosion by water.—A. E. BEITLICH

The valuation of shrinkage numbers. A new comparison for concrete. A. GUTTMANN. *Zement* (Germany), March 1930, V. 19, No. 12-13, p. 267-9, 305-6.—Measurements of

volume changes of cement test pieces is carried out in many different ways. Different observers use different sizes of test pieces; apparatus, aggregates, storage, time, temperature and moisture in air vary in all cases. Author designed apparatus for making standard tests of volume changes and examined different types of cement. The "concrete comparator" consists of a table 50 cm. long with a frame with two sliding microscopes (magnification 100 times). The apparatus enables the study of test pieces of various sizes and dimensions with an accuracy of ½μ. The shrinkage of concrete expressed by its "shrinkage number" is due to the specific surface of the test-piece. Formulas are given to calculate the "shrinkage number" for large blocks from small test pieces.—A. E. BEITLICH

Testing apparatus for the water permeability of concrete. ROBERT OTZEN. *Zement* (Germany), March 1930, V. 19, No. 12, p. 274-7.—An apparatus is described for determination of water permeability of concrete. Special features are: (1) water pressure can be kept constant for unlimited time without any necessary operations, (2) amount of water which passes through concrete in main direction and through sides of test piece can be measured numerically, (3) pressure can be varied within wide limits up to 30 atm. and can also be varied at different test places at the same time, (4) concrete cylinders up to 50 cm. diameter and 30 cm. length can be tested.—A. E. Beitlich.

About the setting of cement. (See MATERIALS—CEMENT)

The appearance of cracks at the edges of cement pats during storage in air after water storage. (See MATERIALS—CEMENT)

Reinforced concrete on railways. (See ENGINEERING DESIGN—MISCELLANEOUS)

ENGINEERING DESIGN

BRIDGES

New bridge and dam at Arnprior, Ont. GEORGE S. M. GRIMMER. *Can. Eng.*, March 25, 1930, V. 58, No. 12, p. 421.—Combination concrete bridge and dam was built over the Madawaska river at Arnprior, Ontario, replaces timber structure destroyed by floods in 1928. Spillway dam is of gravity cross section with ogee curved face. It is 500 ft. long and has maximum vertical depth of 23 ft., the downstream toe being provided with apron at deepest section. Fish ladder of reinforced concrete at center of main channel comprises interconnected series of pools 4 ft. by 5 ft. by 4 ft. in depth, stepped at a slope of approximately 15 degrees. Concrete mix for dam was 1:2:4 of coarse but well graded sand and a large aggregate of crushed limestone rock ranging in size to 3 in. This concrete was difficult to handle and for a portion of the work washed gravel, ¼ in. to 3 in. was substituted, which gave greater ease in handling. Test cylinders averaged 3,900 lb. sq. in. at 28 days. Reinforced concrete bridge, 500 ft. long, has 24 ft. roadway with 6 ft. sidewalk along one side. Bridge was designed to carry vehicles up to 20 tons in weight. Design is two-girder deck bridge, 50 ft. span with system of stringers and overhanging floor beams carrying road and walk slabs. Girders are 24 in. wide, 7 ft. in overall depth, reinforced with 21 1¼-in. bars with ½-in. stirrups. For the bridge a mix of 1:2:3 well graded sand and crushed limestone coarse aggregate passing a 1¼-in. mesh was used. Strength tests gave 3,800 lb. per sq. in. at 28 days. Badly fissured rock foundation made dry coffer dams impossible and some concrete was placed through water by means of a tremie. Much of the coffer dam and concrete work was carried on during winter requiring stock piles of aggregates to be heated by

means of steam points. Water and aggregate were heated to between 100-130° F. Dam contains about 3600 cu. yd. and bridge 1500 cu. yd. of concrete. —G. M. WILLIAMS

Reinforced concrete bridge at Conflans. *Conc. Constr. Eng.* (England), April, 1930, V. 25, No. 4, p. 267-269.—New reinforced concrete arch bridge at Conflans-Fin d'Oise, near Paris, is of 413 ft. 5 in. span. Two parallel hollow arch ribs have rise of 54 ft. 6 in.; are each 4 ft. wide and vary in depth from 4 ft. 2 in. at springing to 6 ft. 8 in. at crown. Suspended floor carries a 16 ft. 4 in. roadway and two 3 ft. 3 in. sidewalks. Wind bracing consists of light system of lattice girders connecting two arch ribs.—F. E. RICHART

Aesthetic in bridge construction. "Aesthetik im Bruerckenbau." FRIEDRICH HARTMANN. 1928, Franz Deuticke, Leipzig and Wien, R. M. 17.50. Reviewed in *Zement* (Germany) April, 1930, V. 19, No. 14, p. 340.—Book is one of few good publications on subject of aesthetic in engineering constructions. Much consideration is given to reinforced concrete bridges. The book is of great value to designing architects.—A. E. BEITLICH

New bridge for Kettle Creek Valley, Ontario. WARREN C. MILLER. *Can. Eng.*, April 15, 1930, V. 58, No. 15, p. 481.—Michigan Central Railway has built near St. Thomas, Ontario, to replace old steel trestle, plate girder bridge supported on concrete piers, designed for Cooper's B—70 loading. Concrete towers 90 ft. high and 13 ft. x 38 ft. at base are carried on spread footings.—G M. WILLIAMS

Ingenious construction on Mill street bridge, Watertown, N. Y. E. H. HARDER. *Eng. News-Record*, April 10, 1930, V. 104, No. 15, p. 603-605.—Old three-hinged steel arch bridge crossing Black River on Mill St. in Watertown, N. Y., was replaced with reinforced concrete arch involving interesting problems of design and construction. Old bridge built in 1897 had become inadequate to carry modern live loads and had deteriorated somewhat because of almost constant spray from upstream dam and waterfall. Arch had span of 165 ft. and rise of 15 ft. 3 in. and end shoes abutted against walls of laminated limestone gorge. About 38 ft. below roadway two ledges of more massive limestone formation extended from vertical rock walls toward the middle of the river. Pieces of broken limestone on river bed between these ledges and the frequent floods would have made erection of centering difficult and risky. For this reason, and for economy, old bridge was used to its maximum extent. Old bridge was not strong enough to permit suspension of forms for new concrete arch, but carried five sets of steel reinforcing ribs after removal of old roadway and sidewalk. Consequently it was decided to utilize structural steel reinforcing in new arch and design steel ribs with sufficient capacity to carry weight of forms and rib concrete and still have sufficient surplus of area to act as reinforcement to rib concrete for carrying live load and remainder of dead load. Usually such structural steel ribs are designed as three-hinged arches because of complicated adjustments necessary to make closure at crown and to induce dead load stresses in the members when hingeless steel arch is built out from fixed abutments. To avoid such adjustments and for ease of erection, ribs were suspended at frequent intervals from the old bridge by hangers. After riveting all splices ribs were adjusted to proper height by raising or lowering hanger rods. With ribs thus erected, they were concreted into abutments and upon release of hanger rods swung free as hingeless steel arches. Six figures.—D. E. LARSON

A new arch bridge in Prague. F. EMPERGER. *Tonind. Ztg.* (Germany), March 17, 1930, No. 22, p. 369-372.—Expansion of city of Prague which is divided by the Moldau river, and absorption of suburbs have rendered necessary construction of new bridges that today present a good study in evolution of masonry bridge design. New arch bridge described was first planned in 1911, but not built until 1926. It was completed in 1928. Cost was 1¼ million marks, 5½ times

price estimated in 1911. Structure has overall length of 740 ft. and consists of four three-hinged, hollow spandrel, barrel arches each with clear span of about 150 ft. and rise of about 16 ft. Two river piers are founded on pneumatic caissons. The lower hinges are not at springing but are 15 ft. from bases of piers. Hinges of Mesnager type were formed by crossing heavy rods at hinge point. Full-size tests of hinges showed failure occurred only when steel had reached its yield point. Bridge has 34-ft. roadway and total effective width including sidewalks, of 53 ft. Structure consists of twin arches symmetrical about roadway center line. Water main is carried between these two units. Type of design simplified form work, as one half of arch could be poured and same forms then shifted to other half.—A. A. BRIELMAIER

Handbook of reinforced concrete construction. (See FIELD CONSTRUCTION—BRIDGES)

BUILDINGS

Reinforced concrete jetty and terminus at Cherbourg. *Conc. Constr. Eng.* (England), March, 1930, V. 25, No. 3, p. 193-196.—The new harbor station includes (a) a main building 280 meters long by 42 meters wide, a hall 240 m. long and 15 m. wide covering a traffic approach from the street and (c) a single-span hall 240 m. long and 39 m. wide. These structures are all carried on reinforced concrete piles. The buildings use reinforced concrete construction in many interesting forms. Structural frames for main building are rigid frames and roofs are of reinforced concrete.—F. E. RICHART

The great food storage house in Leipzig (Germany). FRANZ DISCHINGER. *Zement* (Germany), March, 1930, V. 19, No. 12, p. 279-81.—Description of large storage building with two cupolas. Each cupola has span of 76 meters and rests on 8 reinforced concrete columns. Entire building is 280 meters long. Illustrations show every phase in cupola construction and arrangement of reinforcement in concrete. Description is given of static pressure tests on small models. (cf. *Beton und Eisen* (Germany), 1929, No. 18-19.)—A. E. BEITLICH

Reinforcement of a concrete ceiling in the central telephone office building in Breslau (Germany). OTTO VEIT. *Zement* (Germany), April 1930, V. 19, No. 15, p. 354-5.—Concrete ceiling, previously designed and constructed for live load of 250 kg./m² was reinforced to carry live load of 650 kg./m². New constructions were made with high early strength cement.—A. E. BEITLICH

Changes in New York City building code. *Eng. Con.*, April, 1930, p. 162.—Some of changes in Code which affect reinforced-concrete design are: (1) Permission to use special steels, (2) decrease in wind pressures from 30 to 20 lb. per sq. ft., wind loads to be carried in frame, (3) reductions in minimum live loads for many types of buildings, (4) decreases in minimum loads required for light factory buildings, stables and some garages, (5) reductions in live loads required on columns, piers and girders in buildings over five stories high, and (6) increases in working stresses allowable on steel used in reinforced concrete and on steel and concrete columns to conform in general to increases made in permissible stresses for ordinary structural steel.—N. H. ROY

Reinforced concrete arch hangar at Montreal. *Eng. Con.*, April, 1930, p. 180-181.—First application of arch to this type of structure has been made at Point aux Tremble Airport, Montreal, Que. Hangar is 100 by 100 ft. in area with an arched roof and buttressed columns. Arch rings are at 10 ft. intervals. Roof slab is supported on intrados of arch, thereby making formwork for roof continuous on inside. For design, ultimate steel strength was taken as 55,000 lb. per sq. in. and working stress at 16,000 lb. per sq. in. Ratio of moduli of elasticity of steel and concrete was taken as 15. Shrinkage coefficient was taken as 0.0005. Concrete mixes were carefully designed and controlled.—N. H. ROY

Construction anatomy.
(See ARCHITECTURAL DESIGN)

DAMS

Combination arch and gravity dam. J. B. JAKOBSEN. *Eng. Con.*, April, 1930, p. 162.—Professor Probst, University of Karlsruhe, Germany, has designed combination dam in which lower part is gravity dam and upper part is arch dam. Arch dam is supposed to be free to slide at a joint between arch and gravity sections. Design eliminates undesirable feature, usually encountered, of having arch dam act as gravity dam near its base with the consequent high tensile stresses developed due to fixed base. Design is patented in Germany, but is not believed to be in this country.—N H. ROY

Laboratory tests of reinforced concrete arch ribs. WILBUR M. WILSON. *Bul. 202*, University of Illinois, Engineering Experiment Station. Feb. 25, 1930, V. 27, No. 26, 102 p.—Thirteen reinforced concrete arch ribs were tested, each with span of 17 ft. 6 in. and rise of 4 ft., thickness of 5 in. at crown, and of 10.8 in. at springing. Load was applied at six points, in most cases symmetrical but in two tests non-symmetrical. Series included ribs of four different widths. Effect of spiral reinforcement studied in two specimens. Abutments were cast integrally with arches, and were rotated to varying degrees in certain tests. Standard cylinders showed strength of concrete from 1500 to 3600 lb. per sq. inch. These tests indicate that concrete in rib developed same ultimate strength as in 6 x 12 in. controlled cylinders. Wide variations in value of E_c were observed and computed and it was found that this value was greatly affected by concrete-placing operations. All specimens were tested to failure. These tests indicated that strength of arch is determined by its weakest section and that failure to consolidate concrete properly at any section may be very serious. Very close agreement was found between test results and values computed by usual elastic theory, under the assumptions that value of E_c was constant at all sections and that moment of inertia of section be computed on basis that concrete resists tension and that value of E_c is same in tension as in compression. Spiral reinforcement was found not to increase strength of slender arch ribs such as those tested. Time yield of concrete under long continued loading was found to result in rotation of abutments without an increase in the moment but rate of rotation was much less at end of test period than at its beginning. Ribs having ratio of unsupported length to width of 30 were not weakened by any tendency to buckle when not subjected to lateral loads. Numerous diagrams and tables show tests of each arch rib in detail, manner of failure and comparison of principal data of test and of computation.—ARTHUR R. LORD

Dependability of the theory of concrete arches. HARDY CROSS. *Bul. 203.* University of Illinois, Engineering Experiment Station. March 18, 1930, V. 27, No. 29, 36 p.—Mathematical investigation of a large number (150) of arches indicates that over one-half of total stress may be readily and accurately computed by ordinary "elastic" or geometric theory of arches in any of its common forms of expression. Flexural stress due to live load cannot be predicted with absolute precision. Departure of actual stress from computed value in any arch can scarcely be greater than probable variation in quality of concrete and will generally be much less. Results by the elastic theory may be expected to be within 5 per cent of true values. The theory of arches is more dependable than theory of flexure as generally applied to beams and is a safe and convenient guide to design. There appears to be no good foundation for belief that stress at section of weak concrete will be automatically reduced by the presence of such weak concrete. It may even be increased at such sections if elastic properties at several sections vary simultaneously. Variations in E_c from $+50$ per cent to -33 per cent are considered, and many tables and diagrams are given showing the computed effect of such variations at various sections on

moment at all principal sections. Temperature changes are also discussed and computed but effect of rib shortening is not considered.—ARTHUR R. LORD

A method of arch design. G. P. MANNING. *Conc. Constr. Eng.* (England), April, 1930, V. 25, No. 4, p. 245-248. Article is continuation of an analysis of symmetrical arches. Consideration is given to effect of movements of abutments.—F. E. RICHART

Reinforced concrete cement silos. *Conc. Constr. Eng.* (England) April, 1930, V. 25, No. 4, p. 249-255.—Eight new reinforced concrete silos of 2000 ton capacity each are used to store cement at plant of the British Portland Cement Manufacturers, Ltd. at Greenhithe. Silos have inside diameter of 32 ft. 6 in. and height of 96 ft. 6 in. Each silo is carried on 16 columns supported by footings on chalk foundation. Silo walls are 6 to 6½ in. thick, and are reinforced with vertical bars and horizontal hoops. Each silo is divided by vertical walls into four compartments, one semi-circular and three 60-degree sectors. Silos discharge cement in bulk into enclosed steel wagons of 25-ton capacity, electrically driven.—F. E. RICHART

Reinforced concrete tanks. W. S. GRAY. *Conc. Constr. Eng.* (England) April, 1930, V. 25, No. 4, p. 257-264.—Methods of designing open circular and rectangular tanks embedded in ground are given, with assumptions for lateral pressures on sides and uplift on bottom. Moment coefficients for bottoms of circular tanks are given in diagrams. Means of increasing stability of empty tank against floating are discussed.—F. E. RICHART

Design of reinforced concrete slabs. W. L. SCOTT. *Conc. Constr. Eng.* (England), March and April, 1930, V. 25, No. 3 and 4, p. 167-177, 221-231.—This paper is based on the analysis of thin plates or slabs under concentrated loads by M. Pigeaud (cf. *Annales des Ponts et Chausees*, Feb. 1921). Treatment given is for rectangular slabs freely supported along edges and subjected to loads on rectangular areas of varying sizes. For highly concentrated loads transverse and longitudinal moments remain sensibly constant for all panel ratios greater than 2 to 1. While analysis gives smaller moments than older methods, it is contended that it is more closely in accordance with true conditions. Diagrams are given for bending moments in panels of various panel shapes and concentrations of load. Formulae are also given for intensities of shearing stresses along supports and along edges of loaded areas. Second article gives numerical examples of analyses of slabs subjected to one or more concentrations of load.—F. E. RICHART

Marine structures in reinforced concrete. R. N. STROYER. *Conc. Constr. Eng.* (England), April, 1930, V. 25, No. 4, p. 232-244.—Use of floating caissons in harbor and wharf construction, as well as for bridge piers and sea walls is discussed. Methods of building and launching the caissons are given. Many examples of caissons used in wharf and quay walls in various parts of world are shown, with views and diagrams.—F. E. RICHART

Influence lines as deflection diagrams. E. M. SHEPHERD. *Jour. Institution Engineers.* (Australia). Mar., 1930, V. 2, No. 3, p. 73.—For any structure the ordinate of influence line for any particular force (or group of related forces such as moments, shears, etc.) can be given by displacement of points in structure, when some pre-determined displacement is induced in structure, this value being unity on preselected scale. Nature and position of this displacement determines what force will be shown by influence line. Reference is made to D. B. Steinman's statement (cf. *Eng. Record*, May 13 & 20, and Nov. 15, 1916) that influence line for any reaction, shear, bending moment, or stress is identical with deflection diagram produced by unit displacement at point of application, and in direction of desired reaction, shear, bending moment, or stress. Proofs are based on Maxwell's reciprocal theorem. This rule is easily applied but must be correctly interpreted. Various methods of application for structures, simple

framed and otherwise, are discussed, with proofs in an appendix. Principle forms basis of an accurate method for determining influence lines for statically indeterminate structures by mechanical means.—JOHN E. ADAMS

Reinforced concrete on railways. *Engineer* (England). Apr. 25, 1930, V. 149, No. 3876, p. 466.—It was reported at International Railway Congress, Rome, in 1922, that trials of reinforced concrete sleepers were unsuccessful on Italian State Railway, where 300,000 had to be taken up from main lines and put in station yards. The Roman reporters recommended, (a) substitution of hardwood tree-nail for cast iron bush used in Calot sleeper, to take coach screw, (b) interposition of a wooden packing between sleeper and bottom of rail, (c) complete suppression of tamping under centre portion of sleeper. Present report states that many thousands have been laid since that time, on Paris, Lyons & Mediterranean, Nord, and Paris-Orleans Railways, and they appear to be technically satisfactory. On small radius curves, it is found that the Calot sleeper tends to widen its gauge by ¼ in. to $\frac{5}{16}$ in. and then remains constant. When plant for mass production is near a river from which sand and ballast may be taken without transport cost, cost is less than for wooden sleepers. Benefits due to use of quick setting cement and high-tensile steel; use of pin joints to reduce secondary stresses; introduction of systematic deformations before striking centers in bridge construction, are mentioned in this year's report. Greater development in use of concrete is reported for small span railway bridges and road bridges. London & N. E. Ry. favors reinforced concrete for building construction because of durability, low maintenance costs, fire resisting properties, better distribution of load over foundations, and for marine work subject to depreciation by worms. Dense concrete, secured by control of water cement ratio, grading, and regulation of mix, should lead to greater strength, permanence, and serviceability. Although damage due to smoke is reported in several instances, one reporter gives an account of test on the Southern Ry. of Austria on a concrete structure. It was observed that layer of soot was confined to surface; there was no evidence of molecular change in iron; no chemical change in concrete; presence of sulphuric and sulphurous and carbonic acids was not discernable except in outside 1 c. m. layer of concrete. A 3 c. m. thick cover was therefore considered sufficient, provided concrete was compact. —JOHN E. ADAMS

Flat-slab concrete track deck in St. Louis Union Station. W. F. SAUNDERS, JR. *Eng. News-Record*, April 17, 1930, p. 652-654.—Enlargement of union station at St. Louis, Mo., as described in *Eng. News-Record*, Nov. 22, 1928, p. 766, consists of trucking subway with flat slab roof carrying station tracks and platforms. One of many problems in design and layout encountered was that of 21 in. flat slab, 30 ft. span, to carry E-60 loading plus 30 per cent impact. This slab carries tracks across subways and some old basements. It is supported by twin columns centered under tracks wherever possible. Panel lengths were consequently varied to conform to local conditions. Two-way reinforcement was used. Longitudinal (parallel to tracks) and transverse moments were separately determined. Conditions in railroad deck are so dissimilar to those in ordinary flat slabs that it was not deemed advisable to take for granted empirical rules of flat-slab design, but meticulous check of moments was made by method of variable moments of inertia. For interior spans, ordinary formula of center-to-center length minus two-thirds column head diameter, was used, but it was found that for end spans, something nearer to center-to-center span should be used. Some longitudinal moments, figured by use of equivalent uniform loads and by influence lines for separate wheel loads, gave about 5 per cent differences. Other moments, in end spans of odd length, were greatly different. It was consequently deemed advisable to use influence lines for all but typical cases. Full continuity was figured from one end of expansion joint to next. For typical cases, longitudinal positive moments for one track were resisted by "track strip" ("common strip" in

ordinary flat-slabs) 12 ft. wide and centered on track and mid-strip 6 ft. wide. Live load was considered applied over full track strip and transferred in part to the mid-strip. Computation of relative deflections showed track strip carried three-fourths and mid-strip one-fourth of load. Mid-strip was reinforced for negative moment over supports, track-strip for full negative moments at column head. Top steel was run through all spans. Transverse column strips were continuous series of spans and were computed as such. Loading was distributed three-fourths to track-strip, one-fourth to mid-strip. Platform beam reactions were taken as concentrations. In computing longitudinal live and dead load reactions on transverse column strips, effect of continuity was considered. Bending in columns was determined by computing unbalanced dead load moment at column (generally zero, except at end spans), plus live load unbalanced moment for certain position of wheel loads, and proportioning this moment between column and next span of slab in ratio to moment of inertia divided by the length to hinge point. Columns were designed for this moment and direct load.—N. H. ROY

A successful reinforced concrete base for street railway tracks. I. O. MALL. *Concrete*, April, 1930, V. 36, No. 4, p. 13-18.—Development of street railway tracks has, for most part, taken form of "cut-and-try" methods based on judgment of builders, with little evidence that any of various types of track construction have been based on research analysis. Methods followed in development of reinforced concrete track support in Canal Street in New Orleans are based on careful structural design supplemented by thorough tests under conditions simulating actual service. Roadbed structure consists primarily of reinforced concrete foundation to which rails are anchored by means of resilient rail anchorages. Foundation slab forms continuous girder and two longitudinal angles are embedded in concrete beneath each rail. Rail anchorage units are fastened to these angles. Design provides cushion between rails and pavement and also between rails and reinforced concrete base itself. Cushion between rail and pavement along its side consists of ¼-in. coat of asphalt applied to rail before concrete is placed. Transmission of shock from the rail to concrete base is prevented by placing a ¼-in. thickness of elastite immediately under base of rail. Further elimination of impact is obtained by means of specially designed anchor plate and diaphragm. Diaphragm consists of two oval-shaped thin steel plates crimped on outer edges over round steel ring and welded at periphery to form closed steel container. An oval-shaped bearing plate is welded to both top and bottom diaphragm plates. These bearing plates are smaller than diaphragm plates and are concentric therewith. Rail is held in place with clips, anchorage bolts passing through diaphragm and bearing against angles embedded in foundation. Inside of diaphragm contains asphalt - asbestos mixture forced into place under pressure while hot. Asphalt-asbestos cushion gives stability to diaphragm and aids in dissipating vibration absorbed from rail. In testing these anchorages, load of 10,000 lb. was applied suddenly at intervals of 2 seconds over period of 5½ months, making total of more than six million successive load applications, considered to be equivalent of 75 years of actual service. Life of anchorage diaphragms is expected to be commensurate with that of rest of structure.—Six figures.—D. E. LARSON

Cooling towers at Croydon. *Conc. Constr. Eng.* (England), April, 1930, V. 25, No. 4, p. 265-267.—New reinforced concrete cooling tower at Croydon is one of four which will eventually be required for power plant of Croydon Electricity Station. Tower is of hyperbolic type, Mouchel-Hennebique system, and is 128 ft. in diameter and 164 ft. high. Shell of tower will be inclined inward for 75 ft., then will extend vertically and will finish by being inclined outward. Temporary steel scaffolding and steel forms are used in construction.—F. E. RICHART

Design of retaining walls. W. W. HAY. *Concrete*, April, 1930, V. 36, No. 4, p. 22-24.—Formulas and data

usually required by designer of reinforced concrete retaining walls are presented in detail. Formulas are simple in theory and easy to apply, and data are based on years of experience. Information on following subjects is included in text: (1) formulas for most common cases (2) equivalent live loads uniformly distributed (3) economical types of retaining walls (4) toe and heel pressures and (5) foundations, drainage and joints. Three figures and three tables.—D. E. LARSON

Concrete for resisting sea water.
(See PROPERTIES OF CONCRETE)

ROADS AND PAVEMENTS

Typical concrete pavements in British Columbia.
(See FIELD CONSTRUCTION—ROADS AND PAVEMENTS)

ARCHITECTURAL DESIGN

Aesthetic in bridge construction.
(See ENGINEERING DESIGN—BRIDGES)

The "cement-house" in Charlottenburg (Germany). W. V. DEBSCHITZ. *Zement* (Germany), April 1930, V. 19, No. 14, p. 332-4.—"Cement-house" is office of German Cement Syndicate and all possibilities to use cement and concrete in building construction are shown in its exterior and interior. Nearly unlimited are these possibilities for architect, especially the use of glazed colored concrete slabs. Synthetic stones can be made in every desired form.—A. E. BEITLICH

Terrace structures. "Terassenbau." RICHARD DOECKER. 1929, Fritz Wedekind und Co., Stuttgart (Germany), R. M. 15.00. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Feb. 1930, V. 82, No. 9-10, p. 81.—Author describes advantages of terrace construction in building hospitals, hotels, office buildings, apartment houses and smaller buildings. Book discusses construction of healthy building.—A. E. BEITLICH

Construction anatomy. Bauanatomy. W. BRUENING. 159 pages. Editor: Deutsche Bauzeitung G. m. b. H., Berlin. R. M. 12.00. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria) Feb. 1930, V. 82, No. 7-8, p. 64.—Starting from the excavation, the book discusses every step in building construction up to the roof. Special consideration is given to the combination of forms and art to obtain architectonical effects.—A. E. BEITLICH

"Structure" - concrete. *Zement* (Germany), March 1930, V. 19, No. 13, p. 312-5.—Architect has simple means to prepare surface of any concrete structure to obtain wonderful effects. Cement on the surface is prevented from hardening by a treatment with "Contex," a certain lacquer preparation, and can be brushed away. This leaves the texture of the concrete open. Choice of different aggregates, coarse or fine, colored or colorless, enables architect to fit appearance of building for any purpose. Method can be used for bridge construction, industrial and other buildings. Several illustrations show effects of this treatment in practice.—A. E. BEITLICH

Where concrete reigns. HOWARD ROBERTSON AND F. R. YERBURY. *Architect and Building News* (England), Feb. 7, 1930, V. 62, No. 2, p. 197-200.—One of the most interesting new churches in the world has just been completed at Elisabethville, France, a new town on banks of Seine, about 35 kilometers from Paris. Creation of town was due to initiative of a Franco-Belgian group. The problem of providing for devotional needs of population has produced a church entirely unusual in planning, construction, material, and decorative treatment, yet maintaining to a high degree devotional atmosphere. The architect is Paul Tournon, a pre-war graduate from the Ecole des Beaux-Arts, with an extra-

ordinary gift in decorative design. His school work was generally in tree classic —the Beaux-Arts manner of his day. Thus it is interesting to find this winner of academic prizes galore, standing today in ranks of moderns as an innovator, working in unusual forms and unaccustomed materials. Economy was an important factor in Tournon's scheme for Elisabethville. Neither site nor financial appropriation was large.

Plan of church shows its interesting structural basis. Church is 95 ft. by 51 ft. and contains nave 24 ft. broad and 65 ft. high, framed by powerful piers. On either side are aisles which are unusually low; this factor coupled with the great height of the nave, gives fine effect of scale. The architect, realizing the difficulty of obtaining dignity with small plan, utilized height. His structural system is unlike the mediaeval, but is based on reduction to minimum of points of support and envelopment of interior with glass. Great concrete framed, stained glass windows, have framework constructed in a way scarcely possible in any material other than concrete. Main piers of nave are carried up above roof and connected externally by cross beams, which make of them a series of rectangular hoops. From this structural skeleton the vaulted ceiling is suspended. Concrete beams at eaves level and aisle roof level help to support respectively vault and aisle roofs with their lanterns, and connect system of concrete "hoops" in the longitudinal sense. Vaulting in concrete is formed with double envelope and ribs are emphasized by glazed pavement lights, which enhance lighting of interior. Main feature which draws attention is undoubtedly decorative treatment of west front, which forms great panel carved in concrete, the motif of which are central figures of the two Patron Saints of France and Belgium, Jeanne d'Arc and Saint Michael, dominating composition, surrounded by winged figures carrying arms of Belgian provinces and effigies of mutilated French cathedrals. Work was carried out by the sculptor Sarabecolles. It was done *in situ* in wet concrete. Procedure adopted was that previously used in the tower of Villemomble * * * New church tower at Villemomble, France, also by Tournon, is concrete design erected to give distinction to otherwise ordinary structure. Interior armature of tower is in reinforced concrete, but lower portion is encased in brick and rubble stone. Tower is 45 metres high, with upper sculptured portion 16 metres. Method of working sculpture at Villemomble is almost entirely new and demands sculptor of unusual skill, for with work in wet concrete there is no possibility of general retouching, and each section, after being primed, has to be finished in about seven hours. First step was to prepare concrete cone, from models at 1/20 scale, to which was attached metal armature and the rough boasting of figures. The cone is 4½ in. thick, but larger part of the concrete had no metal fixings near the surface, to permit greater latitude for sculptor. Concrete was in proportion of about 8 cwts. of portland cement to 18 cu. ft. of sand, and was built up in courses in usual way, each course being from 4 ft. 10 in. to 6 ft. 6 in. high. Sculptor executed his statues in wet concrete directly from his models and without retouching. Every day a course had to be completed. Concrete was poured at about four in afternoon, and after 18 hours of setting (i. e. at 10 on following morning), the sculptor started work. During next seven hours he established his main forms, after which the concrete was allowed a further day to harden, when detail was finished off.—REXFORD NEWCOMB

FIELD CONSTRUCTION

BRIDGES

The arch-bridge over the river Ammer near Echelsbach (Germany) and the construction material question. KESTLER. *Zement* (Germany), March, 1930, V. 19, No. 12, p. 285-7.—Discussion of building mate-

rials for bridge, construction of which was described in previous article. (cf. *Zement* (Germany), Dec. 1929, V. 18, No. 44, p. 1305-8; *Jour. Amer. Concrete Inst.*, Jan. 1930, p. 19.)—A. E. BEITLICH

Handbook of reinforced concrete construction. "Handbuch fuer Eisenbetonbau." F. v. EMPERGER. 3rd edition, V. 6, Wilhelm Ernst and Sohn, 1930, R. M. 13.60. Reviewed in *Zement* (Germany), April, 1930, V. 19, No. 15, p. 364.—The two parts of vol. 6 illustrate construction of reinforced concrete bridges in all details; they give static calculations and many examples of practice.—A. E. BEITLICH

The new state highway bridges in Austria. ANT. HAFNER. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), March, 1930, V. 82. No. 11-12, p. 85-90.—Description and, discussion of more than 40 highway bridges; mostly reinforced concrete structures. Diagrams and graphs show dimensions and static and dynamic calculations. Photographs of 35 bridges are given.—A. E. BEITLICH

New bridge and dam at Arnprior, Ont. (See ENGINEERING DESIGN—BRIDGES)

Ingenious construction on Mill street bridge, Watertown, N. Y. (See ENGINEERING DESIGN—BRIDGES)

A new arch bridge in Prague. (See ENGINEERING DESIGN—BRIDGES)

New bridge and dam at Arnprior, Ont. See ENGINEERING DESIGN—BRIDGES)

BUILDINGS

The tower of the newspaper "Tageblatt" in Stuttgart (Germany). E. OTTO OSSWALD. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), March 1930, V. 82, No. 13-14, p. 103-4.—Description of a modern newspaper building in reinforced concrete. Since the ground-water contained much gypsum, it was necessary to use a portland cement which was low in CaO contents for the foundation. The latter consisted of 1.5 m. reinforced concrete. The entire superstructure was built in reinforced high early strength concrete.—A. E. BEITLICH.

Concrete in building construction. A. WEISS. *Zement* (Germany), March, 1930, V. 19, No. 11, p. 246-50.—Contribution to historical and technical development of building construction. Describes several methods of reinforced concrete construction, especially of beams, girders, slabs and floors. Use of sliding forms is illustrated for silos and storage bins. Light concrete is used where good heat insulation is necessary and for manufacture of hollow bricks. Concrete is a very good material in hands of an artist to obtain architectonical effects.—A. E. BEITLICH

Reinforcement of a concrete ceiling in the central telephone office building in Breslau, Germany. (See ENGINEERING DESIGN—BUILDINGS)

Terrace structures. (See ARCHITECTURAL DESIGN)

MISCELLANEOUS

Construction control and supervision. "Bauaufsicht und Baufuehrung." TOLKMITT. 1929, 5 ed., 264 p., Wilhelm Ernst und Sohn, Berlin, R. M. 10.00. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria). Feb., 1930, V. 82, No. 7-8, p. 64.—Book is adviser for the contractor on subjects of construction materials, construction machineries, accident prevention and safety devices.—A. E. BEITLICH

The new power station in Uppenborn of the city of Munich (Germany). LUTZ PISTOR. *Zement* (Germany), March 1930, V. 19, No. 12, p. 284-5.—A power station with output of 36,000 h. p. is described. River Isar delivers 200 cu. meters water per second through 9 km. channel to intake. Nearly two million cu. meters ground were moved for channel construction and 100,000 cu. meters were excavated for power house, the construction of which consumed 50,000 cu. meters concrete. Lock was built in

river with 75,000 cu. meters concrete. Surface conditions made it advisable to pour concrete from an elevated distribution arrangement. Cement and aggregates were transported over crown of dam to a 1000-liter mixer and an automatic equipment for measuring materials. Mixer delivered 40 cu. meters concrete per hour to 5 chutes through which plastic masses were poured. Concrete road was built to connect power station with main highway.—A. E. BEITLICH

Winter construction—a question of mortar. FRITZ EISEMANN. *Tonind. Ztg.* (Germany). Apr. 14, 16, 19, 1930, V. 54, Nos. 30, 31, 32, p. 508-10, 527-9, 543-6.—A discussion of economics of winter construction, which costs about 5 per cent more than in summer. Limiting factor is apparently mortar and by using certain precautions including more cement, warm aggregates and mixing water, $CaCl_2$ and protection for freshly built wall, very satisfactory work may be done.—F. O. ANDEREGG

Cost keeping for reinforced concrete contractors. A. E. WYNN. *Conc. Constr. Eng.* (England), March, 1930. V. 25, No. 3, p. 200-212.—A detailed analysis of plant costs and charges, including items of overhead, depreciation, repairs, interest, storage, insurance and taxes. Forms are given for monthly expense schedule.—F. E. RICHART

Transit system of mixing concrete. D. H. BURROWES. *Str. Eng.* (England), April, 1930, V. 8, No. 4, p. 131-137.—Method of making and supplying mixed concrete known in United States and Canada as transit system employs special concrete mixer mounted on truck body in which accurately proportioned batches are hauled dry until shortly before delivery. Water is admitted and mixing commenced a few minutes before delivery, providing freshly mixed concrete to job. Mixer unit and batching plant are described together with the operation of system and license to operate. Advantages are given concerning control of mix and quality of concrete. Economics of system, and advantages to contractor are also pointed out.—V. P. JENSEN

Concrete foundations for the Siemens (Railway). *Tonind. Ztg.* (Germany), March 17, 1930, No. 22, p. 376-378.—Problem of transporting 60,000 employees has led Siemens-Werke to construct special railway from plant in Berlin-Siemenstadt to junction with main railway lines of city of Berlin. In foundation work, chemical process was used which, by means of sunken pipes, petrifies sandy soil. Process, controlled by "Tiefbau und Kalteindustrie A.-G.," Berlin, was used to insure stability of some existing bridge piers in Spree river, next to which new caissons were to be sunk to lower elevation. Pipes were inserted through old piers to sandy stratum below. Compound having silicic acid as one of its constituents was pumped into bearing soil. Second and different solution was then forced through pipes. Chemical reaction between two solutions results in petrified sub-soil. Same process has been successfully used at Klingenberg Power Development, at Dusseldorf Water Works, and on other works.—A. A. BRIELMAIER

Lochaber hydro electric power undertaking. *Engineer* (England), Apr. 25 and May 2, 1930, V. 149, No. 3876, 3877, p. 456, 479.—For purpose of producing electrical energy for production of aluminum, a watershed of 303 sq. miles in Scotland is being enclosed. Project includes dam 550 ft. long by 42 ft. high; tunnel 16 ft. in dia. and 4 miles long, connecting Spean reservoir with Loch Treig; dam at outlet end of Loch Treig to maintain proper water level; weir 900 ft. long and 30 ft. high to raise level of Spey river; conduit 3 miles long; power house at Fort William. Work was first commenced on power house and tunnel between Loch Treig and Fort William. Tunnel is 15 miles long and 15 ft. diameter. Provision for electrical supply, sanitation and hospital accommodation for 3000 workmen had to be supplied, and 23 miles of 3 ft. gauge contractor's railway was built to connect various camps. Tunnel was driven from 22 faces simultaneously, through grey and red granite, mica

schist, flagstones, red porphyry etc. Plants were installed at surface at each operating point. To reduce skin friction losses, tunnel was lined with 4 to 1 concrete. Steel strutting was employed and lining effected in six sections: first two haunches were carried up to 3 ft. above final floor level; sides were put in to about 12 ft. above floor; ceiling was then put in, shuttering being carried on travelling carriages; finally, floor was constructed in "Ciment Fondu." Concrete was mixed in electrically driven mixers and cement, sand and spoil from excavating were carried to and from working faces with battery operated electric locomotives. Aggregate was formed of crushed stone from excavations and sand was obtained from pits near mouths of some of adits. For intercepting streams, some of which are fast running, weirs were formed in dams well above bed. Water from streams is delivered into tunnel by means of concrete lined shafts, divided into two tubes, one to liberate air entrained by water. Before blasting inlet at Loch Treig end of tunnel, bulkhead of concrete, flat faced in upstream side and convex downstream, was recessed in concrete lining to tunnel. A 4 x 4 ft. opening was made in it and steel safety plate held in position over it by bolts and nuts. After blasting with 4000 lb. gelignite, water was allowed to drain away through safety opening, and bulkhead subsequently removed.—JOHN E. ADAMS

Ready - mixed concrete. (See PROPERTIES OF CONCRETE)

ROADS AND PAVEMENTS

Machineries and mechanical installations for modern road construction. E. SEILER. *Teer und Bitumen* (Germany), March, 1930, V. 28, No. 7, p. 97-106.—Description of different kinds of crushers, mills and dryers for road construction materials. Mixers for macadam and concrete are discussed. Special consideration is given to power consumption and economy. Different methods for construction of concrete roads and finishing their surface follow. General discussion of American and German methods.—A. E. BEITLICH

Cement-bound roads. R. A. B. SMITH. *Conc. Constr. Eng.* (England) March, 1930, V. 25, No. 3, p. 184-192. —Cement bound road is modification of water-bound macadam road in which strong matrix binds stones together and gives an impervious surface. It is constructed by placing 2-in. layer of stone macadam and rolling lightly, then spreading 1-in. layer of 1:2 sand mortar, then another 2-in. layer of stone. Road is then rolled until macadam is consolidated and mortar is brought to surface. Rolling should be done within 1¾ to 2 hr. after mortar is placed. Characteristics of finished road are similar in many ways to those of concrete roads.—F. E. RICHART

Typical concrete pavements in British Columbia. *Can. Eng.*, April 1, 1930, Vol. 58, No. 13. —During past five years Vancouver has placed 350,000 sq. yd. of concrete pavement, this type being used on a majority of recent jobs. Grandview Highway is paved with 20-ft. slab 7½ in. thick at center, 9-in. at edges, mix 1:2:3, ⅜-in. expansion joints at 40 ft. intervals with dummy joints to care for contraction midway between expansion joints. Burnaby section is 18 ft. wide, 7 in. and 9 in. thick, mix 1:2:3½, expansion joints at 60 ft. and dummy joints at 20 ft. intervals. Dummy joints consist of ¼ in. premolded filler pressed into 2 in. deep slot made at the surface by leg of T-iron. Irregular cracking has been entirely eliminated by these joints together with longitudinal center joint. Quality of concrete materials were carefully watched and slump limited to 2 in., resulting in a 4500 lb. concrete. Concrete was water cured by spray from nozzles placed at regular intervals.—G. M. WILLIAMS

SHOP MANUFACTURE

Homogeneous coating of construction materials with lead. A.

KARSTEN. *Zement* (Germany), March, 1930, V. 19, No. 10, p. 232-4.—Method is given and an apparatus described to produce homogeneous coating of lead on concrete tiles and other construction materials. The lead is heated in gun with oxyacetylene flame but is not affected by any combustion gases. Coating can be made 5 mm. thick.—A. E. BEITLICH

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